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STRUCTURAL DETAILS OF THE NEW REINFORCED CONCRETE BRIDGE AT GRAND RAPIDS, MICH.

By Wm. F. Tubesing.*

GENERAL DESCRIPTION.—There has recently been completed across the Grand River at Grand Rapids, Mich., a new street bridge of reinforced concrete which deserves to be classed as a model structure of its kind. This bridge was designed and built under the direction of Mr. L. W. Anderson, City Engineer, and it consists of five arch spans, one 87 ft., two 83 ft. and two 79 ft. long. Fig. 1 shows an elevation and longitudinal section of one of the arch spans, and Fig. 2 shows details of the spandrel and parapet wall construction and of the railings. The main dimensions and general structural features of the bridge can be determined quite clearly from these drawings.

LOADS AND CONDITIONS.—The loading and other conditions assumed in designing the bridge were as follows:

Dead load.	Lbs. per cu. ft.
Concrete	150
Earth filling	120
Pavement, 12 ins. deep	150
Live load.	**Lbs. per sq. ft.**
Center, 20 ins. of roadway	250
Remainder of roadway	150
Sidewalks	100

For concentrated load on the roadway the designer assumed a 15-ton steam road roller 11 ft. between axles, with 6 tons on the forward wheel 4 ft. wide and 4½ tons on each of the two rear wheels 20 ins. wide and 5 ft. apart on centers. The assumed moduli of elasticity were 1,500,000 for concrete and 30,000,000 for steel. The maximum compression allowed on the concrete in the arches was, excluding temperature stresses, 500 lbs. per sq. in., and, including temperature stresses due to a variation of 40°, 75 lbs. per sq. in. The maximum tension allowed in the concrete in the arches was, including temperature stresses due to a variation of 40°, 75 lbs. per sq. in., and, excluding temperature stresses, 50 lbs. per sq. in. The maximum shear allowed was 75 lbs. per sq. in. Concrete slabs, girders, beams, floors, walls and posts were required to have a safety factor of 4 in one month.

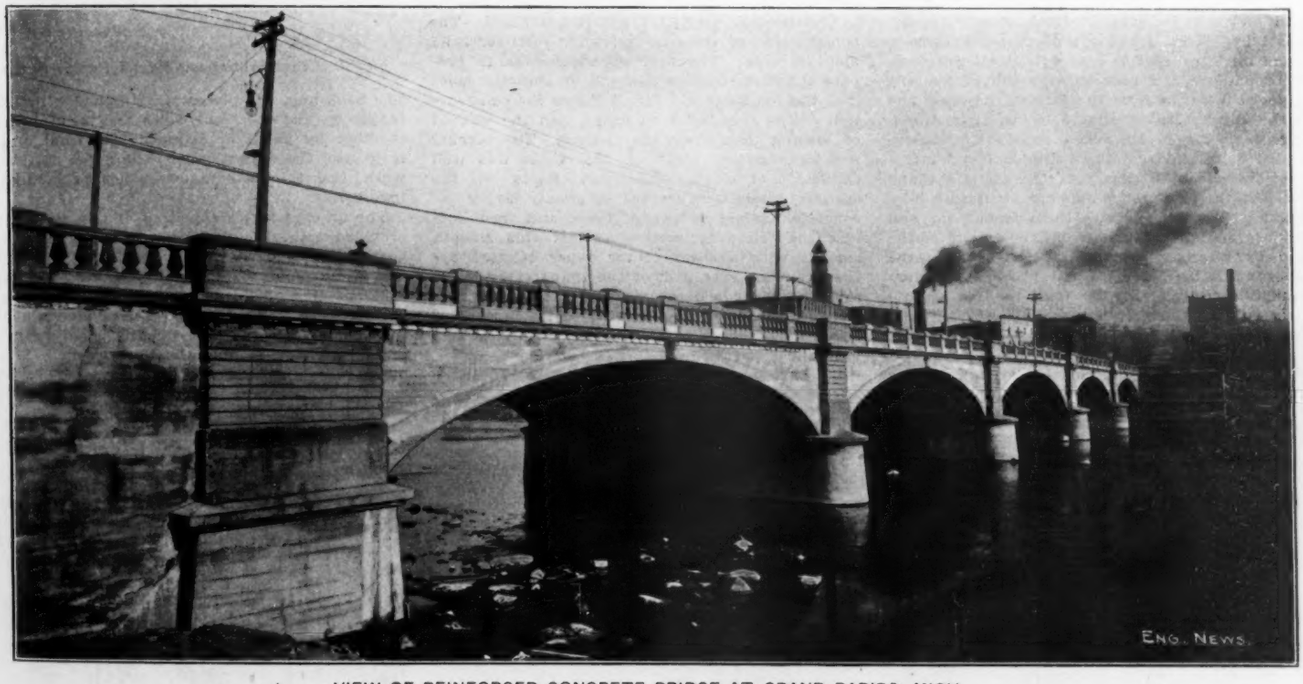
It was required that steel ribs under a stress not exceeding 18,000 lbs. per sq. in. must be able to take the entire bending moment of the arch without aid from the concrete and have flange areas of not less than the 1-50th part of the total of the arch and 1-50th of the crown. The actual stress in the steel, when embedded in and acting in combination with the concrete, was required not to exceed 20 times the allowable stress in the concrete. In slabs, girders, beams, floors and walls subjected to transverse stress, the steel was assumed to take the entire stress without aid from the concrete. The imbedded steel in walls and posts subjected to compression only was used as a precaution against cracks due to shrinkage.

SUBSTRUCTURE.—The substructure of the bridge consists of four piers and two abutments of the general construction shown by Fig. 1. Both piers and abutments rest upon a rock bottom of limestone; to get to solid rock excavation had to be carried through hardpan containing granite boulders. Work on the excavation for the piers was begun June 17, 1903, and both piers and abutments were completed to the springing line level on Nov. 24, 1903. There were about 2,000 cu. yds. of excavation, and 4,000 cu. yds. of concrete work involved. The approaches to the abutments were made box-shaped, of concrete, reinforced by Johnson bars. The space between the west approach walls was filled with stone and gravel, and that between the east approach walls was utilized for a public comfort station.

Two grades of concrete were used for the abutments. For the lower portion of the abutments the concrete consists of 1 part natural cement, 2½ parts sand and 4 parts gravel, and for the upper portion the concrete is composed of 1 part Portland cement, 3½ parts sand and 7 parts gravel. The last mixture is also used for the piers, spandrel and retaining walls and the facing of abutments.

ARCH RING CONSTRUCTION.—Preparatory

*City Engineer's Office, Grand Rapids, Mich.



VIEW OF REINFORCED CONCRETE BRIDGE AT GRAND RAPIDS, MICH.
L. W. Anderson, City Engineer; Joseph P. Rusche, Contractor, Grand Rapids, Mich.

to the construction of the arch rings, eight spans of an old steel bridge had to be removed, cleaned and painted, and a foot bridge had to be built. Work was begun at the east end of the bridge, and as fast as the old steel spans were removed the piling for the centering was driven. Owing to the hardpan bottom these piles would not penetrate more than 6 ins. in any place. This made it necessary to brace the piles very thoroughly, as shown by the drawings of the centers given in Fig. 3. These drawings give all dimensions, and explain the construction of the centers clearly. It will be noted that wedges formed the means provided for lowering the centering.

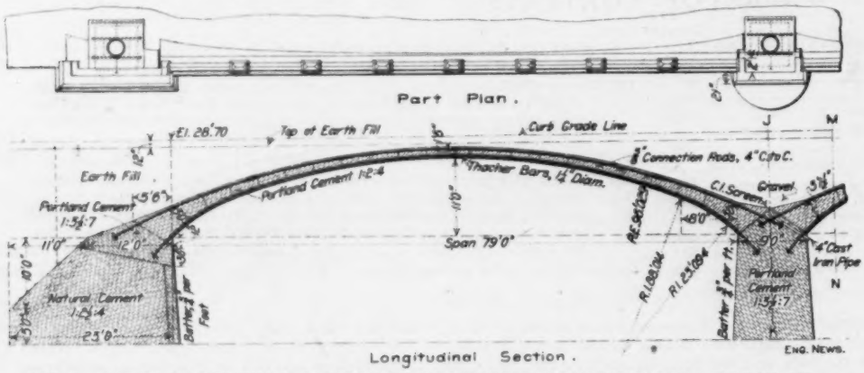


FIG. 1. PART LONGITUDINAL SECTION OF REINFORCED CONCRETE ARCH BRIDGE AT GRAND RAPIDS, MICH.

The dimensions and thickness of the arch rings for the 83-ft. spans are shown by Fig. 3, and Figs. 1 and 2 show details of one of the 79-ft. spans. The concrete used for the arch rings was composed of 1 part Portland cement, 2 parts sand and 4 parts gravel. Each ring was reinforced by 1¼-in. Thacher rods in pairs, consisting of an intradosal and an extradosal rod each. The outside pair of rods on each side of the ring is placed 3 ins. in from the face of the arch, and all the other pairs are spaced 14 ins. apart transversely of the bridge. The rods are also embedded 3 ins. in from the extradosal and intradosal faces of the ring. At the abutment and pier ends the reinforcing rods are fitted with 3-in. washers and nuts, to give them anchorage, and they are made continuous from end to end of the span by means of turnbuckles. Generally the rods came in lengths of 27 and 33 ft. Connection through the arch ring between the upper and lower rod of each pair was made every 4 ft. by means of a ⅝-in. rod, provided with a hook at each end.

The method of constructing the arch rings was as follows: The endmost sections of the reinforcing bars, which had been anchored into the piers and abutments during their construction, were bent down to the curve of the arch ring and connected up with the arch rods proper, as shown by the view Fig. 4. Scantling placed transversely across the lagging of the centers served to block up the soffit rods, while the upper rods were held in place by the connecting rods with eye hooks previously described. No trouble whatever was experienced in placing the rods and keeping them accurately in position. The concrete was mixed fairly wet, and was worked around and underneath the rods with 1x3-in. tampers in much the same way as ballast is worked under railway ties. After embedding the lower bars, a stiffer concrete was deposited and rammed in 6-in. layers. The arch ring was built in transverse sections, and each section was completed in a continuous operation in one day. The crown section was built first, and then the two skewback sections, and last the intermediate sections. To define the sections during construction a form normal to the soffit was employed. It required five days to construct each arch ring. The first arch was begun May 25, 1904, and the last was completed on Aug. 4, 1904.

SPANDREL WALL CONSTRUCTION.—The construction of the spandrel walls calls for brief mention only. The first operation was to prepare the arch ring for the wall by chipping holes into the top surface. Fig. 3 shows the construction of the spandrel wall forms, and the method of bracing them from the centers. The lagging for forming the faces of the walls was well chalked and oiled. Expansion joints in the spandrel walls were formed by simply laying the concrete against a vertical form, and then butting the following section against this smooth surface with a sheet of tar paper inserted between. The lower part of the spandrel walls inside and the whole top surface of the arch ring were waterproofed by a mortar coat composed of 1 part Portland cement, ½ part thoroughly slacked lime and 3 parts sand.

RAILINGS AND ORNAMENTAL WORK.—The railings and ornamental parts of the bridge, such as keystones, brackets, consoles, dentiles and panels, were cast in molds and set in place much as cut stone would be. Special molds were employed for each of these different shapes. These molds were plastered with an earth damp mortar composed of 1 part cement and 2½ parts fine sharp sand, which was followed up with a backing of wet concrete composed of 1 part cement, 2 parts sand and 3 parts broken stone passing a ¾-in. ring. The facing mortar was made 1½ ins. thick. The castings cannot be told from dressed stone at a few feet distance.

The part elevation and sections in the drawings of Fig. 5 show the arrangement of the various castings to form the completed railing, coping, etc. To specify, A is the arch ring, B the brackets, C the coping, and D, E and F, respectively, the base, balusters and rail of the bridge railing. The blocks G and H show the keystone and railing post. The forms or molds for each of these parts are shown by the other drawings of Fig. 5. A description of each of these forms follows:

The keystones were molded in wooden forms, consisting of one piece, a, forming the top and front; of two side pieces, f; of a bottom consisting of two parts, b and c, and of a back piece, g

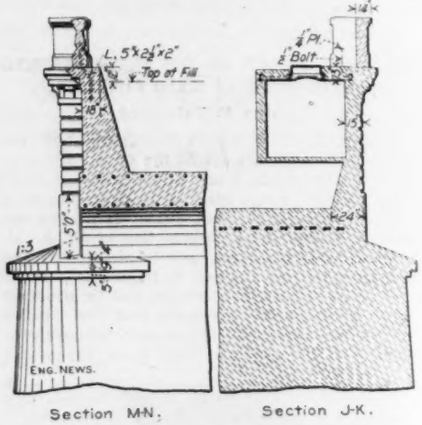


Fig. 2. Cross-Sections of Grand Rapids Bridge.

The back and side pieces are stiffened with 2x3½ in. pieces, and the front, sides and back are held together by yokes or clamps. The front of the mold was the only portion calling for particular work, and this was made of boards laminated together.

The bracket molds consisted of two side pieces provided with grooves for receiving the front and back pieces, and with slats for tie rods clamping the whole mold together. It will be noted also that the side pieces had nailed to them inside a beveled strip to form a groove in each side of the cast block. The purpose of this groove was to provide a bond to hold the bracket more firmly in the adjoining concrete of the wall. The bottom of the mold was formed by a 2-in. plank, and when the concrete had been tamped in place the forms were removed, and the bracket was left on the bottom to set. It may be noted here that a goodly number of the brackets showed a crack at the joint marked x caused by tamping at the point y. In construction the bracket castings were set at proper intervals on the spandrel walls.

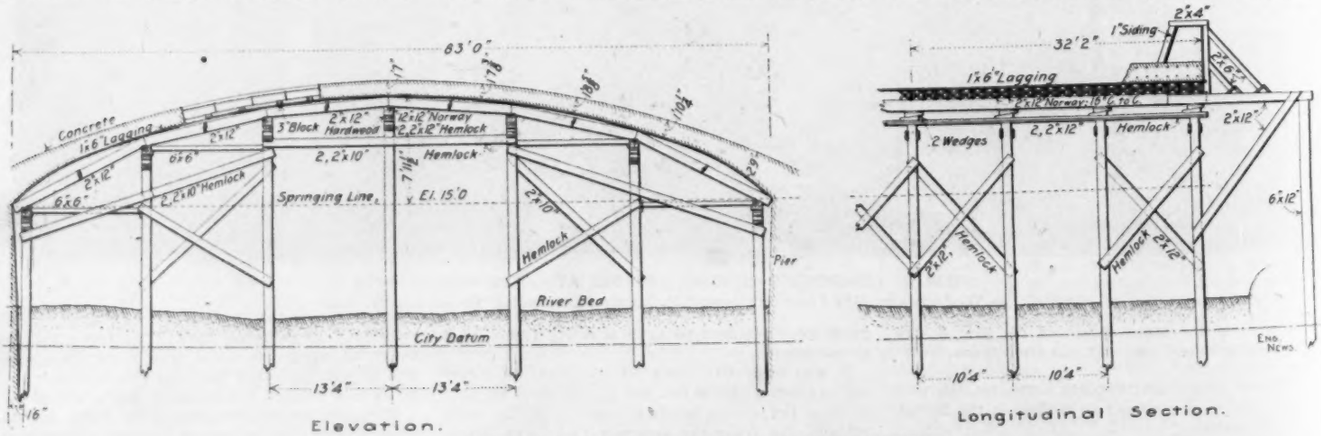


FIG. 3. CENTER FOR 83-FT. SPAN, GRAND RAPIDS BRIDGE.

which has been completed up to the level of the line X Y. The coping course was then built up around the bracket blocks to the level of the bottom of the railing base.

The mold or form for the coping course was designed to build the coping in successive sections,

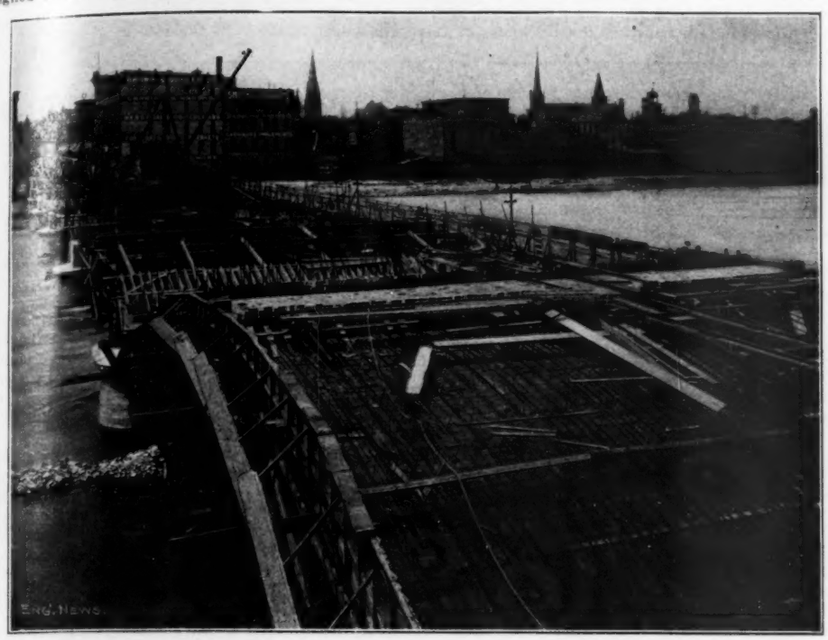


FIG. 4. VIEW OF GRAND RAPIDS BRIDGE DURING CONSTRUCTION, SHOWING REINFORCEMENT IN PLACE ON CENTERS.

and was built up around the bracket blocks, and supported from the centres as shown by the drawings. To form the expansion joints in the coping course there were inserted across the mold at proper intervals a short iron plate ¼ in. thick, cut to fit. The cutting of this plate was found to be a slow operation.

The forms for the base of the railing (section D) consisted of 1¾-in. stock for the sides, and ¾-in. stock for the slopes. They extended across the arch, and were held together by a very simple though very efficient clamp. This consisted of two 2x3x33-in. pieces nailed to a 2x3x17-in. piece by means of galvanized iron strips. About halfway down the long pieces, a ½-in. rod was run through, and secured up against blocks, h, placed about 56 ins. apart. These blocks were removed as the concrete was put in place. It will be noticed from the cross-section of the railing that the balusters are set into sockets formed in the top of the base course. These sockets were formed by means of the mold shown at W and Z.

In casting the balusters, Section (E), a ⅜-in. cast iron mold, consisting of four iron sides and an iron top, was used. Originally there were two end plates of iron, but it was found more convenient to have the bottom one of wood and allow the cast spindel to stand and set. The mold was held together by ½-in. bolts. It would have been more practical to have had the side casting composed of two parts.

The form for the railing is built up around the tops of the spindels. The bottom piece is 1x9 ins., to which 4¼-in. ogee molding is nailed. The sides are of 1-in. stock, and are clamped together. The top is finished off with a trowel.

The mold for the posts is made in four parts, which fit together at the top and bottom by a bevel joint, as shown in the one-fourth section. The broad sides rest against the narrow ones, and are held against the same by means of ½-in. rods running through 2x3 in. stock; 2-in. projections of the broad sides facilitate the removal of the form from the completed post.

As regards future work of the character described here the following suggestions may be made:

Be sure you have driven your piles deep enough to be certain they will not settle. A half section of the arch ring had to be chipped out by hand for the reason that the piles settled when the load was put on, which would not have occurred if the piles had received a few more blows. It took three days to chip out the same with a good force of men. Don't be too slack with tamping. Be certain that your forms are well braced, and thus insure yourself against any bulging. Aim to get as good a carpenter as you can for your special work.

AN INTERESTING STEEL ARCH BRIDGE DESIGN.

The great masonry dam, now nearly completed, forming the closure of the New Croton Reservoir of the water-supply of New York City will contain, as an accessory structure, a large steel arch bridge of interesting design. Bids for the construction of this bridge have already been called for, and it is expected to have the bridge completed by March 1, 1905. From the contract drawings on which bids are asked we select some representative plans and details, reproduced herewith. These show the intended construction completely; the following explanatory information, drawn from drawings and specifications, may facilitate the comprehension of the design:

GENERAL CONSTRUCTION.—The view, Fig. 1, is a front elevation of the New Croton Dam, looking upstream, i. e., toward the east. The spillway, at the left or northerly end, is a masonry overfall weir about 1,000 ft. long, curved upstream to an L-shape in ground plan and discharging into a sloping rock channel leading past the main part of the dam to the valley below. The driveway running along the top of this dam is to be carried over this spillway channel by the arch bridge in question, a structure 200 ft. in span. At one time a stone masonry arch was contemplated for this crossing, occupying the same position as shown in Fig. 1 for the steel arch. The general dimensions of the steel arch design, as rise, span and width, were fixed by the provision which had been made for the stone arch. The bridge (see also Fig. 2) is 200 ft. in span c. to c. of bed-plates, has a rise of 43 ft. 5¾ ins. from center of bed-plates to center of rib at crown, and is 18 ft. 6 ins. wide c. to c. of ribs. The total rise from water table to roadway is 58 ft.

The arch is of the unbraced or solid-rib type, as appears from Fig. 3. The two ribs are of box-girder section, with a depth varying from a maximum of 42 ins. at the crown to 36 ins. at the abutments. The ribs rest against the skewback masonry by riveted shoes bolted rigidly to the masonry; they are therefore fixed-end arches. At the crown, however, a 10-in. joint-pin is provided. In addition the ribs are spliced by riveted connections at the crown. The following clause in the specification, which does not necessarily refer to arch ribs, may indicate the reasons for the compound pin-and-riveted crown joint:

> All field joints and connections shall be thoroughly riveted up at such a stage of erection that all members are truly lined up and free from internal strains and twists when swinging the bridge free.

As the bedplates of the ribs are cast solid with cement mortar, after the ribs have been connected up, there are thus virtually three hinges during erection, which will ensure the absence of initial stresses.

The two arch ribs are set in vertical planes. They are braced together by a full system of four-angle lattice struts and crossed four-angle diagonals. The panel-points of this lateral system correspond with the roadway posts.

The roadway is a horizontal plate composed of transverse 15-in. I-beams spaced 4 ft. 6 ins. apart,

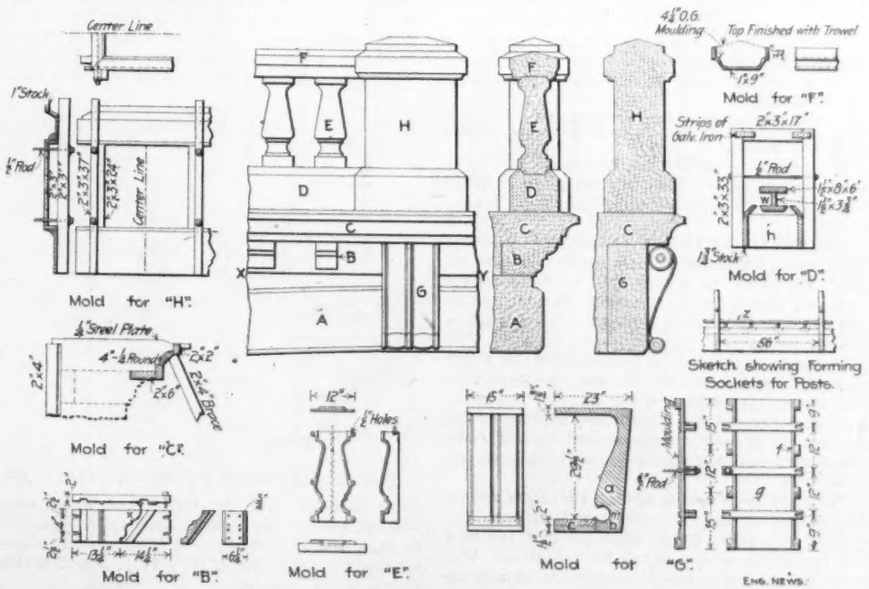


FIG. 5. DETAILS OF SPECIAL FORMS FOR RAILINGS AND ORNAMENTAL PARTS.

and concrete arches turned between these beams. The beams themselves are supported by two longitudinal arcades of riveted construction, continuing downward in the form of built-up steel posts to a footing on the arch-rib. The posts are of box section and the arches connecting their

upper ends are composed of two side-plates joined by a curved channel along the intrados; the top of these arcades is open.

The roadway is provided with an independent system of laterals, consisting of crossed single angle diagonals, 3½ × 3½ × ⅜-in., lying just below the plane of the top of the arcades. The panel division corresponds with the spacing of the posts.

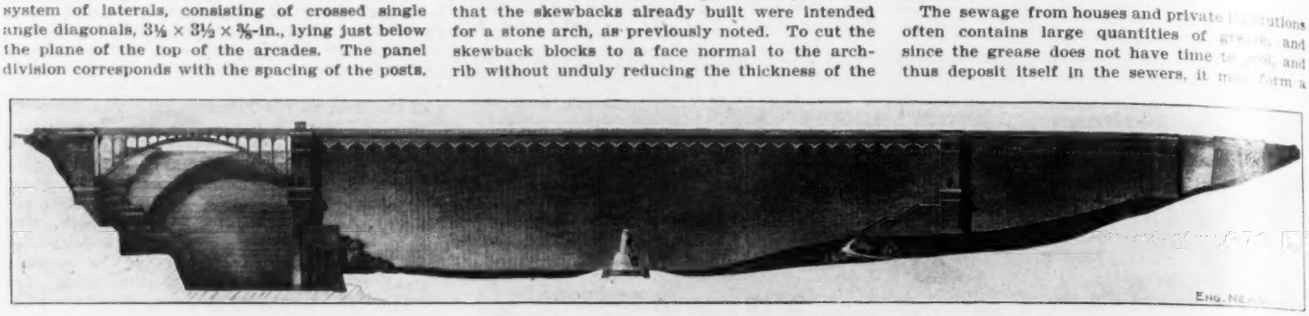


FIG. 1. VIEW OF NEW CROTON DAM FOR THE WATER SUPPLY OF NEW YORK CITY, SHOWING 200-FT. STEEL ARCH OVER THE SPILLWAY.

In line of each post is a four-angle latticed strut in the plane of this top-lateral system and these struts are also the upper members of the sway-frames. A sway-frame is fitted at each post. Fig. 4 shows the general arrangement of the sway bracing.

PAVING AND BALUSTRADE.—The concrete floor of the roadway is covered with two layers of roofing-felt laid in asphalt. On this is a paving of asphalt blocks. The joint between the roadway of the bridge and that of the approaches is covered by a 15 × ½-in. checkered plate. A concrete curb borders the roadway.

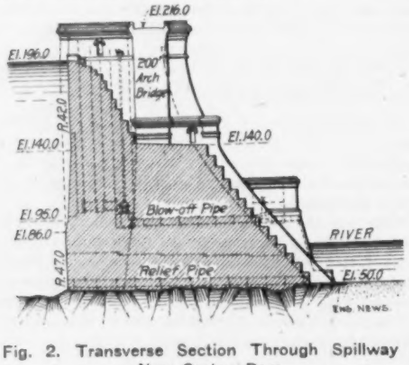


Fig. 2. Transverse Section Through Spillway of New Croton Dam.

The balustrade (Fig. 4) is a continuous hollow rampart built up of plates and angles and capped with a cast-iron top rail. The specification requires that

> the balustrades shall be thoroughly lined up and shall be straight and they shall be tight to prevent rattling as well as leaking.

The plates composing the balustrade are ¼ in. thick, except that a ⅜-in. bottom plate is called for, which is to be "fully spliced to act as lateral chord."

MATERIAL, LOADING AND DETAILS.—The specifications call for a single grade of structural steel, of 56,000 to 64,000 lbs. ultimate tensile strength, 33,600 lbs. elastic limit, 26% elongation and 50% reduction of area. A softer steel is specified for rivets.

The arch as a whole is proportioned for a live load of 2,000 lbs. per lin. ft., and the dead load was assumed at 8,000 lbs. per lin. ft. The roadway is designed for a distributed loading of 100 lbs. per sq. ft. and a concentrated loading composed of 15-ton road rollers having 10 tons on one axle.

The plans show splices in each rib at the middle of the half spans. A space is left between the ends of the upper flange plate so that the web and bottom chord splices can be riveted up. The splice for the top flange plate is made by a plate spanning the gap and fastened to the flange plate (between the flange-angles) by machine screws which are then riveted over cold. Presumably a similar construction is to be followed in the crown joint.

The upper edge of this splice-plate is calked to prevent water leaking into the arch-rib. Also as a protection against water, the foot of each post is to have the pocket formed on the upper side filled with asphaltic concrete.

The details of the footing of the arch-ribs on the skewback masonry were governed by the fact that the skewbacks already built were intended for a stone arch, as previously noted. To cut the skewback blocks to a face normal to the arch-rib without unduly reducing the thickness of the blocks made it necessary to offset the seat as is shown in Fig. 5. The bed-plates, 5 ft. × 3 ft. 3 ins. × 1½ ins., are set in cement.

The development of the æsthetic features of this bridge merits attention, since a prime object in the design was to secure a pleasing appearance. A solid-rib arch of this kind is usually pleasing by its general form and requires only attention to the details to avoid clumsy or inharmonious effects. The complicating condition, in the present case, of having skewbacks already built to receive a bridge of another material and of other form, seems to have given no trouble; at any rate the result in this point is very successful.

Some of the details which were evidently dictated by æsthetic considerations are: First, the design of the posts which carry the roadway. These are not only of varying spacing, widening out toward the abutments, but corresponding to the wider spacing and to the greater height, are made of larger section as the abutment is approached. The central post is 10 ins. wide, while the end post is 14 ins. wide. The effect of this is apparent from Fig. 1. Second, the posts are constructed with recessed edges, to give a pilaster effect, and the projecting portion is carried up to the roadway, so that the posts project in relief from the secondary arches. They are finished at the top with a cast-iron cap. Third, the balustrade is made solid instead of following the open-railing style; this gives a massive cornice effect above the posts and secondary arches. Fourth, the arch-ribs are increased in depth toward the middle of the span. It will be noted that with the exception of the cast-iron post caps there is no item of the bridge which is not essentially a structural element. The details of this bridge were worked out by Mr. Albert Lucius, M. Am. Soc. C. E., under the direction of the Aqueduct Commissioners of the City of New York, Mr. J. Waldo Smith, Chief Engineer.

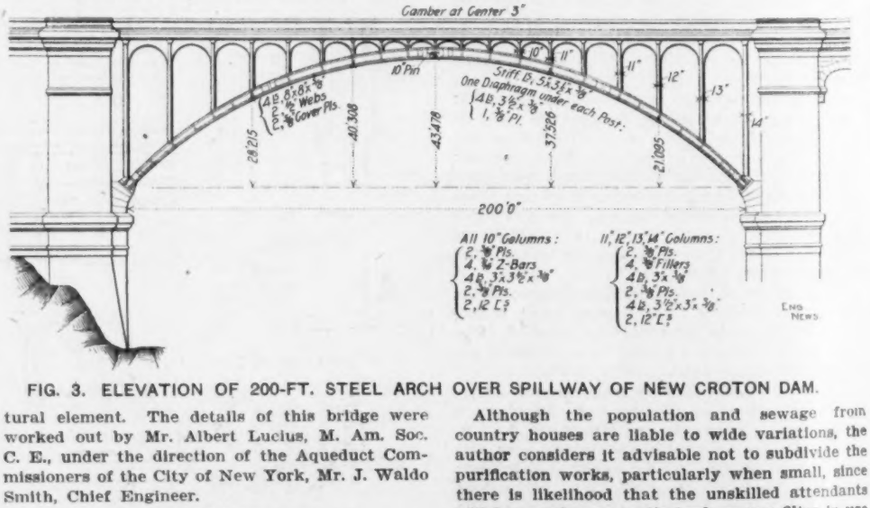


FIG. 3. ELEVATION OF 200-FT. STEEL ARCH OVER SPILLWAY OF NEW CROTON DAM.

SEWAGE PURIFICATION WORKS FOR SMALL POPULATIONS.

Starting with the proposition that the sewage from a single house is often more difficult to treat than that from a large town, Mr. A. J. Martin, Assoc. M. Inst. C. E., of London, England, presented a suggestive paper before a recent meeting of the Institute of Sanitary Engineers. Some of the most significant ideas advanced were as follows:*

The sewage from houses and private institutions often contains large quantities of grease, and since the grease does not have time to cool, and thus deposit itself in the sewers, it may form a coating on the filtering material of the purification works and thus lessen bacterial action. This suggests larger grease traps at buildings. Institutional sewage is also liable to contain large quantities of soap. This absorbs the free oxygen of the sewage and coats the filtering material with greasy and offensive slime.

As a rule septic tanks for small works should be relatively larger than those for large works, since the sewage arrives at the tank with its solids less broken down, both physically and bacterially, than in the case of large works with extensive pipe systems. Moreover, the contents of a small tank are liable to serious disturbance, owing to the sudden displacement of a large percentage of their contents when a bath tub or a wash tub, or even a water closet, is discharged.

One of the most serious difficulties encountered in dealing with sewage from houses and institutions is caused by the heavy grade required for self-cleansing velocities in small sewers with light flows. To lessen the necessary grade the author has in several cases liquefied the sewage at the head of the outfall, thus making it possible "to carry the effluent to filters at any desired distance in a sealed main without appreciable fall."

Another and a great difficulty under which small works labor is the lack of trained attendants, or even those of ordinary common sense.

The requirements for a sewage purification plant for "a mansion or public institution" are summed up by Mr. Martin as follows:

> It should be unobtrusive, and capable of ready concealment in case of need.
> It should give off no perceptible smell.
> It should be simple, automatic, and as far as possible foul-proof.

Although the population and sewage from country houses are liable to wide variations, the author considers it advisable not to subdivide the purification works, particularly when small, since there is likelihood that the unskilled attendants will have only one septic tank or one filter in use when there should be two, or vice versa.

For final treatment filters are generally advisable. Sprinkling feeds to such filters are important aids to purification, but may not be per-

*The complete paper is printed in "The Public Health Engineer" (London) for Oct. 15, 1904.

missible near houses, owing to the liability of odors from septic tank effluents when the latter are sprayed into the air. To avoid this risk a contact bed, either singly or as a preliminary to a percolating filter, may be used. The author prefers fixed to moving distributors, for small installations, since the latter require attention to keep them moving.

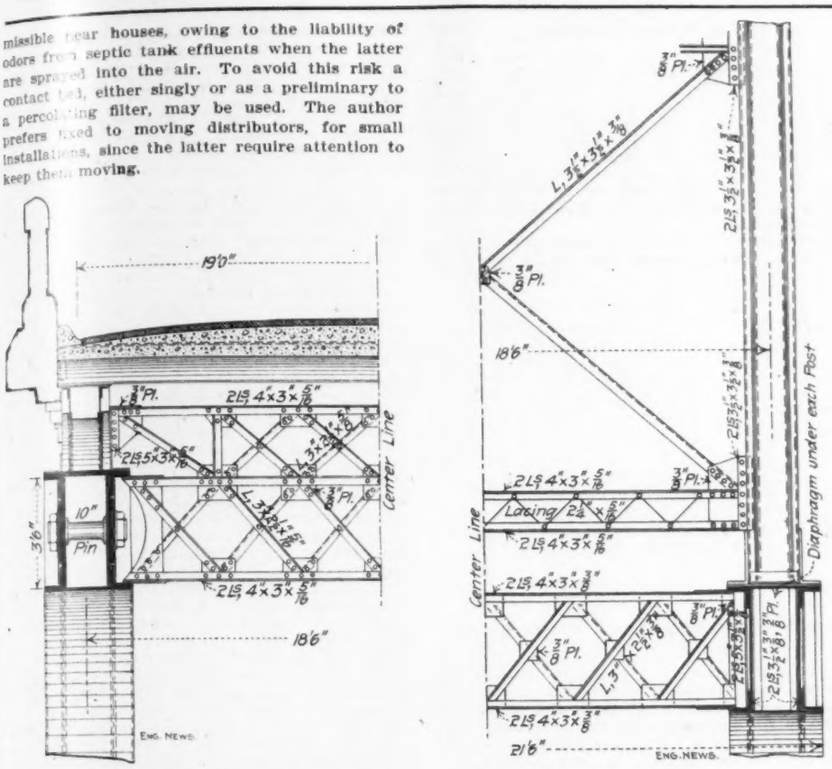


FIG. 4. CROSS-SECTIONS OF 200-FT. STEEL ARCH OVER SPILLWAY OF NEW CROTON DAM.

To avoid short cuts through percolating filters, a contact layer, controlled by automatic valves, may be placed beneath the percolating filter.

In the long run owners will save money by having the designers of purification works superintend their construction and visit them at intervals after they are put in operation.

THE COST OF ONE AND A HALF MILES OF 6-IN. WATER PIPE.

By Edward H. Cowan.

The accompanying table shows the cost of laying about 1½ miles of 6-in. cast-iron water pipe. The work was done during last August and September, there being few rains, and bracing being required in only a few spots.

Trenches were 5 ft. 2 ins. in depth, and the digging varied through all degrees, from the softest spading to the hardest of picking. Taken as a whole, however, the material excavated may be considered as "average earth" without serious error.

The efficiency of the foremen and men may also be considered as average. A contractor who could have spent all his time on the work would doubtless have cut down the expense of the labor somewhat.

All backfilling was done by hand, and the trenches were flushed with water, no tamping being done except around gate boxes. No material was hauled away.

The pipe used was the United States Cast Iron Pipe & Foundry Co.'s standard water pipe, weighing nominally 34 lbs. per ft. The 4-in. hydrant connection pipe weighed 22 lbs. per ft. The hydrants used were the Coffin Valve Co.'s standard 5-in. gate hydrant, having two 2½-in. ports. The Rensselaer Mfg. Co.'s standard double disk gate valves and Buffalo pattern gate boxes were used.

The lead was "Omaha desilverized," and the dry jute gasket packing was a cheap variety, which answers as well for this purpose as the more expensive.

The cost of teaming includes only the hauling of pipe, specials and hydrants. All other hauling, including gates and gate boxes, lead and jute, moving tool boxes, etc., was done by the water company's horse and wagon. The pay of the driver is included in the item "cost of labor," but no allowance is made for the horse and wagon.

Statement of Cost of Labor and Materials for Laying 7,899 ft. of 6-in. Cast-Iron Water Pipe.

	lbs.		
Weight of pipe, per ft.	33.74		
Weight of specials, per ft.	0.67		
Weight of 4-in. hydrant connection pipe, per ft.	0.45		
Total weight, per ft.	34.86		
Cost of pipe, per ft.		$0.4049	
Cost of specials, per ft.		0.0184	
Cost of 4-in. hydrant connection pipe, per ft.		0.0075	
Total cost of pipe and specials per ft.			$0.4308
Cost of hydrants, per ft.			0.0658
Cost of gates and gate boxes, per ft.			0.0544
Weight of lead, per ft.	0.736		
Cost of lead, per ft.			0.0632
Weight of jute packing, per ft.	0.0684		
Cost of jute packing, per ft.			0.0026
Cost of labor, per ft., $0.1867 to $0.2583, averaging			0.2109
Cost of teaming, per ft.			0.0094
Cost of miscellaneous items, per ft			0.0075
Total cost of labor and materials, per ft.			$0.8146

Unit Prices (Including Freight).

Pipe, $24.00 per net ton. Specials, 2¾ cts. per lb.
Hydrants, $26 each.
Gates, $12.60 each. Gate boxes, $3.09 each.
Lead, $4.51 per 100 lbs.
Jute packing, 3¾ cts. per lb.
Labor, per day of ten hours: 1 foreman, $2.50; 2 subforemen, $2.00; pipe handling gang, $1.75; digging and back-filling gangs, $1.50; 1 water boy, $1.
Teaming, 49½ cts. per net ton.

RECENT EXPERIMENTS ON SCREW PROPELLERS AT THE U. S. MODEL BASIN.*

By D. W. Taylor, Naval Constructor, U. S. N.†

The experiments which are the subject of this paper have been under contemplation at the experimental model basin for some time, the design, construction, and testing of the necessary apparatus and appliances having necessarily extended over a considerable length of time. The experiments themselves are quite recent, the first regular test of a model propeller having been made in August last.

The primary object of the experiments was the determination of the power and efficiency of model propellers of the ordinary three-bladed type throughout the range of pitch ratio and blade area likely to be encountered in practice. In view, however, of the present tendency towards increasing speed of revolution of propellers, the experiments were extended to the unusually low pitch ratio of 0.4.

The series of experiments covered 30 model propellers. They were all 16 ins. in diameter, three-bladed, of uniform pitch, diameter of hub 3⅛ ins., thickness of center of blade at the root 9-32-in., at the tip 3-32-in. The pitches used were six in number, namely, 6.4, 9.6, 12.8, 16, 19.2, and 24 ins. For each pitch five widths of blade were used, the mean width ratios, actual total blade area and blade area expressed as a faction of the disk area, being as indicated in the accompanying table. Fig. 1 shows the blade area outlines used.

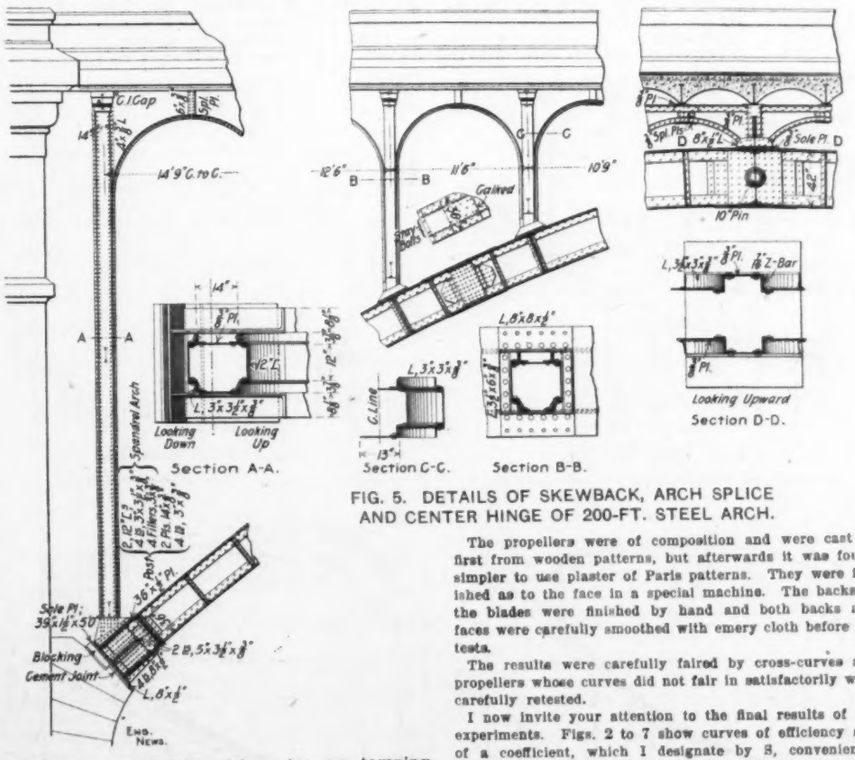


FIG. 5. DETAILS OF SKEWBACK, ARCH SPLICE AND CENTER HINGE OF 200-FT. STEEL ARCH.

The propellers were of composition and were cast at first from wooden patterns, but afterwards it was found simpler to use plaster of Paris patterns. They were finished as to the face in a special machine. The backs of the blades were finished by hand and both backs and faces were carefully smoothed with emery cloth before the tests.

The results were carefully faired by cross-curves and propellers whose curves did not fair in satisfactorily were carefully retested.

I now invite your attention to the final results of the experiments. Figs. 2 to 7 show curves of efficiency and of a coefficient, which I designate by S, conveniently called the power coefficient, for each of the thirty propellers. If P denote the horse-power absorbed by the propeller, d the diameter of propeller in feet and V the

*A paper read before the Society of Naval Architects and Marine Engineers, New York, Nov. 18.
†U. S. Navy Yard, Washington, D. C.

speed of advance of the propeller in knots, we have

$$P = .0093648\ Sd^2\ V^3.$$

Evidently then curves of e, or efficiency, and of S, completely characterize a given model propeller. The above formula will apply to a propeller of any size similar to the model propeller, provided the well-known law of comparison connects the operations of the small and large propellers. We must look upon the application of the law of comparison to propellers with some suspicion until it is practically demonstrated by comparison between model propeller experiments and full-sized propellers, but in the present condition of the science it is our most reliable guide.

The 16-in. propellers experimented with are materially larger than any previous model propellers concerning which I have knowledge. Froude's model propellers, experimented with about twenty years ago, were 8.16 ins. in diameter. Prof. Durand's experiments, described in Vol. 5 of our Transactions (1897), were made with propellers 12 ins. in diameter. A 16-in. propeller tested as these were is by no means a toy. Each propeller was tested up to a thrust of 150 lbs. or a slip of 40%. The propellers of the coarser pitch did not show 150 lbs. thrust with 40% slip, but they did show more than 100 lbs. at the highest speed experimented with. The finer pitched propellers reached 150 lbs. thrust at slips materially less than 40% for the higher speeds of advance.

After the runs the propellers were carefully tested for variation of pitch. In no case was there any appreciable distortion of pitch. The five propellers of pitch ratio 1.0 were also tested for pitch in the machine with a weight of 50 lbs. hung to each blade from its center at a radius of two-thirds the maximum radius of the blade. Some of the narrow blades bent a good deal under this test, but did not change pitch appreciably at any point and took no permanent set.

Table of Propeller Variables.

Propeller number.	Pitch Ratio. .4	.6	.8	1.0	1.2	1.5	Mean width ratio.	Total developed area. Sq. ins.	Developed area ÷ Disc area.
1	6	11	16	21	26	.075	24.28	.1207
2	7	12	17	22	27	.125	40.46	.2012
3	8	13	18	23	28	.200	64.74	.3220
4	9	14	19	24	29	.275	89.02	.4427
5	10	15	20	25	30	.350	113.30	.5635

In connection with Figs. 2 to 7 it may be well to point out the fundamental elements affecting the powers and efficiencies of propellers. They are the diameter, the pitch, the blade area and the shape of blade section. The latter is seldom referred to or included as a fundamental feature in propeller design, but these experiments indicate clearly that it is a thing which must be reckoned with, and that probably many unsolved puzzles in connection with propellers have been dependent upon shape of the blade section. The conventional meaning of the word "pitch," as applied to a propeller of uniform pitch, is the pitch of the driving face or helicoidal face. The back of the blade, however, also has a pitch at each point, and in the conventional section it varies from point to point. If the water always remains in contact with the back of the blade as well as the face of the blade, it would seem obvious that the pitch of the back of the blade necessarily affects the operation of the propeller. It is not generally realized how widely the pitch of the back of the blade varies from the face. For the finer pitch screws of the same section these variations are more pronounced; for the coarser pitches they are not so pronounc... For propellers of .200 mean width ratio and a uni... pitch on the face of 16 ins., the pitch of the back va... from 7.8 ins. to 26.4 ins. Conventionally we call the ... h of this propeller 16 ins., but it is obvious that it ... tically impossible to know what the real pitch ...

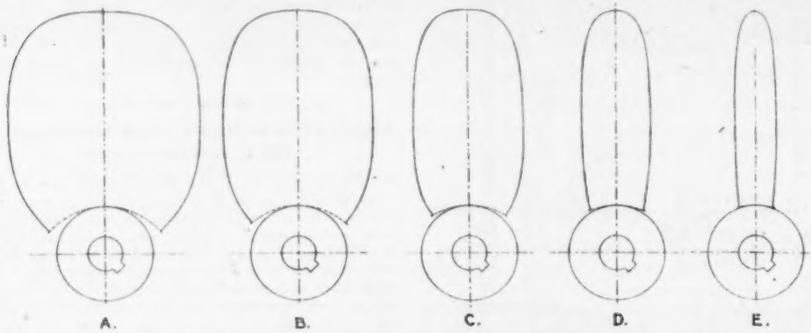


FIG. 1. OUTLINES OF PROPELLER BLADES USED IN EXPERIMENTS AT U. S. MODEL BASIN.
Mean Width Ratios: A = .350, B = .275, C = .200, D = .125, E = .075.

While it is the primary object of this paper ... be-fore the society actual results of comprehensi... experi-ments which each member may analyze for ... self, rather than to set forth theories of my own, ... h to direct attention to a few features of these res... which appear worthy of remark and requiring explana... In the first place we have been accustomed for ma... years to suppose that the maximum efficiency of the ... peller varied but little from 70%. These experiments ... show a maximum efficiency of propeller No. 28 of 78.8%. While the efficiencies do not take account of the hub action, and are hence slightly higher throughout than if the hub action had been considered, I am confident that ... high efficiencies were actually obtained. It will be noted that these model propellers are most accurately finished and the blades are comparatively thin, the thickness at the root being .018 and at the tip .006 of the diameter. There is a steady falling off in efficiency as the pitch ratio is decreased. This was to be expected from theoretical considerations.

Perhaps the most remarkable feature of these results is the disclosure of the fact that as the pitch ratio is decreased the influence of the area of blade upon the results become progressively less until for the propellers of very fine pitch ratio the narrow blades actually absorbed the greater power. For the propellers of 0.6 pitch ratio, curiously enough, at a slip of 26%, it makes practically no difference what the width of blade is, the same power is absorbed by the propeller, although the narrow blades show somewhat greater efficiency. Below this slip the narrower blades absorb the greater power, just as for the 0.4 pitch ratio, while above this slip the wider blades take more power. As pitch ratio is increased, this action becomes less and less, until for the higher pitch ratios experimented with the power absorbed is greater the wider the blade. The increase in the power with increasing width of blade, is, even for the high pitches, however, much less than it should be according to propeller theories of which I have any knowledge. Fig. 8 shows for 25% slip for the series of 1.5 pitch ratio a curve of power absorbed plotted over mean width ratio.

Another remarkable feature is the fact that as the pitch ratios of the propeller decrease the power absorbed and efficiency at low slips are both very great, the slip being based upon a conventional pitch.

Thus considering the series of propellers from Nos. 6 to 10 it is seen that for the narrowest blade the efficiency at zero slip is 66.8%, while the maximum efficiency of this propeller is but 67.4%. As the blade width is increased this effect is decreased. Of course an explanation of

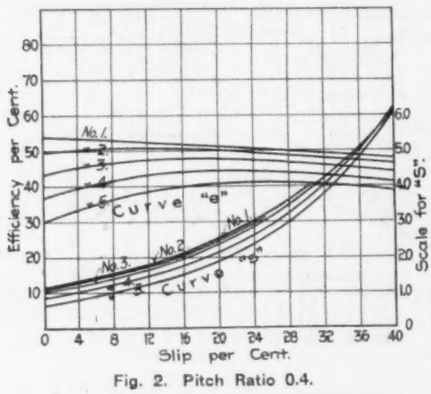


Fig. 2. Pitch Ratio 0.4.

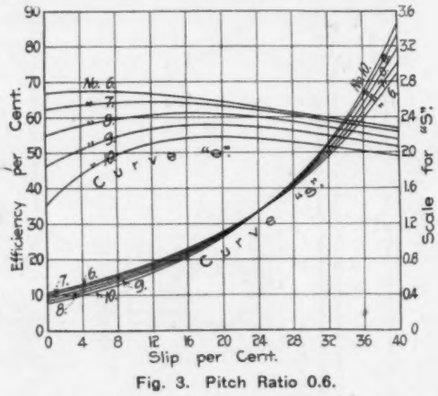


Fig. 3. Pitch Ratio 0.6.

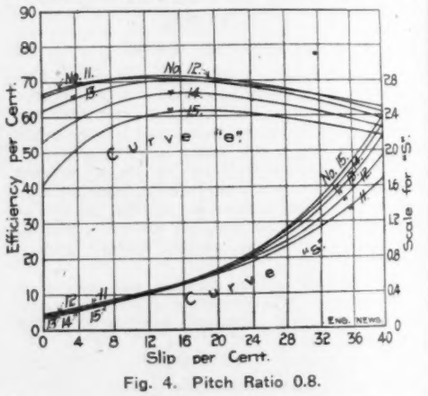


Fig. 4. Pitch Ratio 0.8.

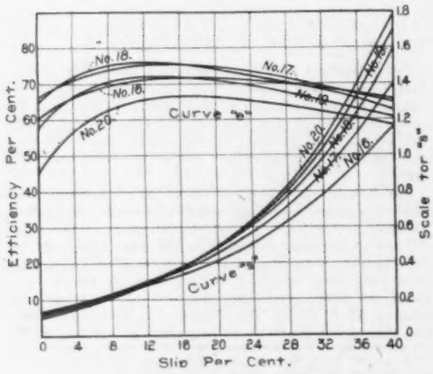


Fig. 5. Pitch Ratio 1.

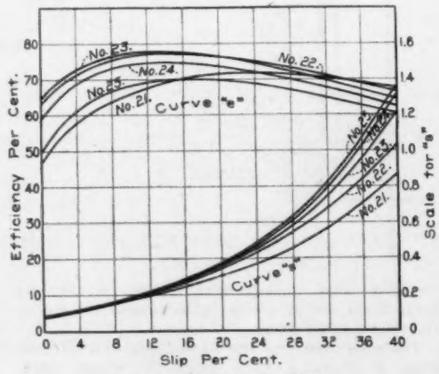


Fig. 6. Pitch Ratio 1.2.

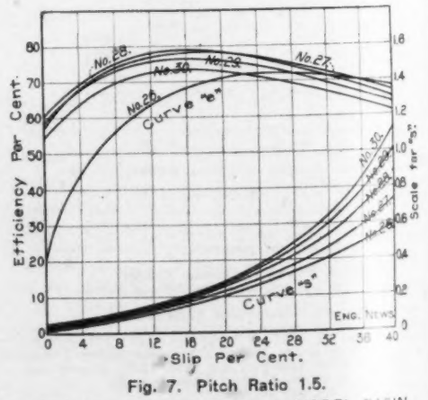


Fig. 7. Pitch Ratio 1.5.

FIGS. 2 TO 7. CURVES OF EFFICIENCY (e) AND POWER COEFFICIENT (s) FOR THREE-BLADED PROPELLERS TESTED AT U. S. MODEL BASIN.

many of the features above described is the fact already pointed out that the slip based upon conventional pitch is necessarily erroneous. It would seem that the real or effective pitch of the propeller was a quantity which varied very much. For small slips the effective pitch would seem to be very much greater than the nominal pitch, whereas for large slips it would seem to become less again. In other words, at small slips the propeller acts as if the following portion of the back of the blade was predominant, whereas at large slips it acts as if the leading portion of the back of the blade assumes predominance.

CAVITATION.

It is generally recognized that model propeller experiments are of little value as regards cavitation owing to the fact that model propellers working under the combined pressure of the air and water have a virtual submergence very much greater than that of full-sized propellers. For many of the propellers experimented with

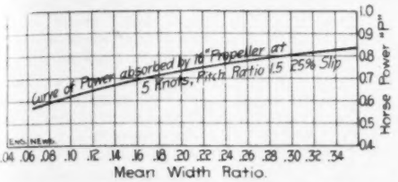


Fig. 8. Power Absorbed at 5 Knots by 16-in. Propeller of 1.5-Pitch with 25% Slip.

there seemed to be a tendency towards reduction of thrust and torque when tested at the 7-knot speed, but there was no pronounced cavitation observed, except in the case of No. 1 propeller when tested at 7 knots. Most of the propellers of this pitch ratio were tested up to 6 knots only. Fig. 9 shows curves of thrust and torque plotted upon slip for propeller No. 1 for 5, 6 and 7 knots speed of advance. The 5-knot curves are normal. The 6-knot curves show evidences of cavitation at about 0% slip and 115 lbs. thrust. The 7-knot curves show strong evidences of cavitation at about—15% slip and 80 lbs. thrust.

It will be noted that cavitation occurred at a very low thrust per square inch of projected area, about 4.3 lbs. for the 7-knot speed, and that the thrust at which cavitation became marked was about 40% greater at 6 knots than at 7 knots. I feel very confident that in the case of this particular propeller the breaking down, which we call cavitation, while affected by the thrust, was strongly affected by two other factors, namely, the speed of revolution and the shape of the blade section. In other words, in this particular case the breaking down of the propeller was a case of cleaving the water as when one draws a stick through water, and at a sufficiently high speed of revolution the propeller would have shown cavitation with almost no thrust. It seems reasonable to suppose that all cavitation is largely of the same nature and a function not only of the thrust but of the speed of revolution and shape of blade section, and that it could probably be mitigated in many cases, or deferred, by modifying suitable the blade section.

As bearing upon the effect of shape of blade section, I submit Figs. 10 and 11. These refer to three propellers, Nos. 23, 23a and 23b. No. 23 was one of the regular series, 16 ins. diameter, 19.2 ins. pitch, 64.7 sq. in. blade, and .200 mean width ratio. Nos. 23a and 23b. had the same diameter and blade area, but had sections as shown in Fig. 10. The pitch of the lines A A for 23a was 19.2 ins. In other words, the face and back of the blade were similarly convex. For 23b the pitch of the portion a-b of the face of the blades and the portion c-d of the back of the blade was the same, namely, 19.2 ins. Fig. 11 shows together the curves of efficiency and S for these three propellers. It is seen that for propeller 23a there is a slight gain of efficiency as compared with No. 23, the usual shape, but a marked falling off in the power utilized. At the slip corresponding to maximum efficiency 23a can utilize only about 50% as much power as No. 23. Propeller 23b shows a falling off in efficiency as compared with 23 and 23a except at very low slips, but has somewhat more capacity for utilizing power than No. 23. Propeller No. 23 was also run reversed, with the backs of the blades driving. The results are not shown on the curves, but the maximum efficiency shown was not as great as 50%.

Fig. 11 illustrates strongly what is evident from careful examination of Figs. 2 and 7, namely, that the shape of blade section is a very important factor in propeller performance. It should be noted in this connection that fine pitched propellers should have as high a value of S as practicable in order to keep down the diameter and increase the pitch ratio.

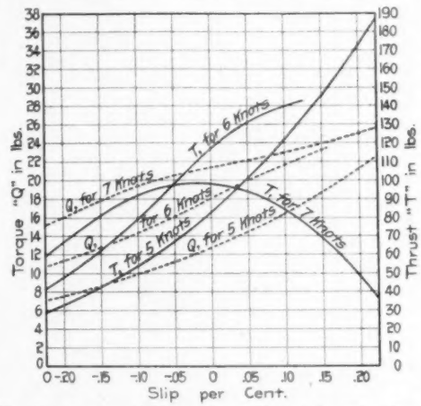


Fig. 9. Curves of Torque and Thrust for Propeller No. 1 at 5, 6 and 7 Knots.

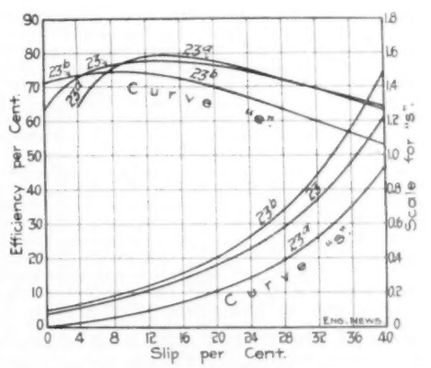


Fig. 11. Curves of Efficiency (s) and Power Coefficient (e) for Three Propellers.

Diameter, 16 ins.; pitch, 19.2 ins. Total blade area, 64.7 sq. ins. Mean width ratio, .200.

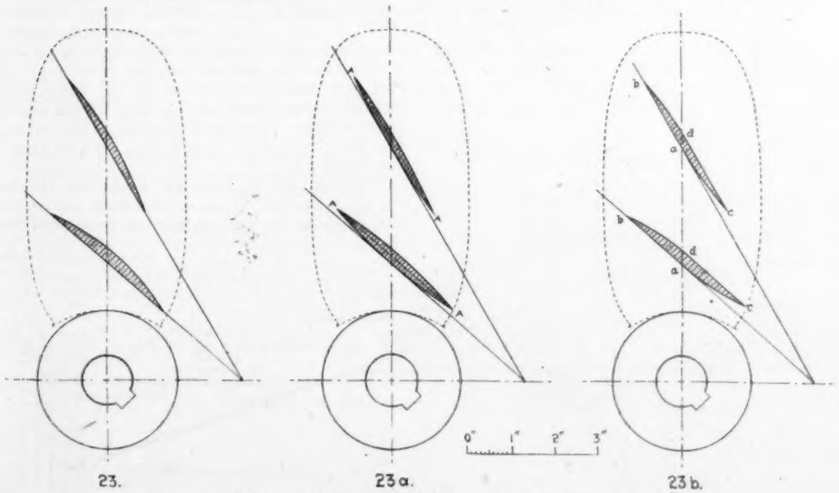


FIG. 10. SHAPES OF SPECIAL PROPELLER BLADES.

COMPARATIVE COST OF FAST AND SLOW FREIGHT SERVICE.*

By W. B. Poland.†

For many years the relative economy of running slow freight trains with engines loaded to maximum capacity, compared with fast freight trains of less than the maximum capacity of engines, has been under discussion. For the purpose of collecting some data on this subject, records were kept on the Indiana Division of the Baltimore & Ohio Southwestern Ry. for the months of January and February, 1904, as follows: 1, For common freights scheduled to make about nine miles per hour, rating being assumed as full capacity of engine for this speed. 2, For semi-quick dispatch freights, it being intended that this class of freight should make an average schedule speed of 15 miles per hour, rating of engines being assumed at 72% of rating for common freights. 3, Quick dispatch freights, it being intended that this class of freights should make an average schedule speed of 19 miles per hour, rating of engine being assumed at 60% of rating for common freights.

The data collected for each of these classes of trains for separate trips was as follows: 1, Temperature; 2, number of engines; 3, number of cars; 4, number of 1,000-ton miles, no arbitrary allowance being included; 5, running time between terminals; 6, rate per hour; 7, total time between terminals; 8, rate per hour; 9, cost of operation; 10, cost of fixed charges; 11, total cost; 12, cost per 1,000-ton miles; and 13, per cent. hauled of the maximum rating for common freight.

The "cost of operation" for each trip was obtained from actual cost of wages, overtime, cost of helper engine and fuel, as shown by records of that trip; the number of engine miles per trip multiplied by the total cost per mile, as shown by monthly records for the following items: Running repairs, classified repairs, oil, sand, waste miscellaneous engine supplies and roundhouse attendance, which amounts to 14 cts. per mile for the 1,500 class engines and 8 cts. per mile for the 100 class engines.

The writer has estimated the average original value of the 1,500 class freight engines at $11,000 each and that of the 100 class engines at $9,000 each. An annual allowance of 5% on the value should be made for interest and 5% for depreciation, making a total of 10% per annum. In order that the use of an engine may be justified, it must earn 10% per annum of the capital invested, and must for each hour of its time be worth a proportionate amount to the railroad company. For the 1,500 class engines this would equal about 12 cts. per hour and for the 100 class engines 10 cts. per hour. This is the fixed charge made against each train for each engine for each hour in transit. The writer has assumed the average value of a freight car to be $800 and on the same basis a car, in order that its use may be justified, must earn 10% of its value per annum, which would amount to $0.01 per hour. On this basis the fixed charges for the train equipment for any trip would be $0.01 multiplied by the number of cars in the train, multiplied by the number of hours between terminals; plus $0.12 or $0.10 multiplied by the number of engines multiplied by the number of hours between terminals.

The percentage of rating handled by each train was obtained from a tonnage statement, by dividing the actual ton mileage made by the ton mileage which would have been made had maximum common freight rating, corrected for temperature, been used. The "total cost" used does not represent the actual cost of handling the train between terminals, but includes only those principal items which vary with, or are dependent upon, each trip. The cost obtained by dividing the total cost by the number of 1,000 ton-miles is therefore correct for comparative purposes only.

The above data have been shown graphically on the accompanying diagram as follows:

First: A chart has been made by plotting the actual running time in miles per hour on the horizontal ordinate and the cost per 1,000 ton-miles on the vertical ordinate. The curve of average results indicates the cost per 1,000 ton-miles corresponding to given speeds in actual running time.

Second: A chart has been made by plotting the actual running time in miles per hour on the horizontal ordinate and the percentage of the maximum rating for common freight on the vertical ordinate. The curve of average results indicate the percentage of maximum rating which must be given trains in order that their average speed may be a certain running time in miles per hour.

Third: A chart has been made by plotting the actual running time in miles per hour on the horizontal ordinate

*Abstract of a paper published in Bulletin No. 56 of the American Railway Engineering & Maintenance of Way Association.

†Late Superintendent of the Baltimore & Ohio Southwestern Ry.; now in charge of the construction and operation of the Alaska Central Ry.

and the time between terminals in miles per hour on the vertical ordinate. The curve of average results indicates the relation between the actual running time and the schedule speed in miles per hour between terminals.

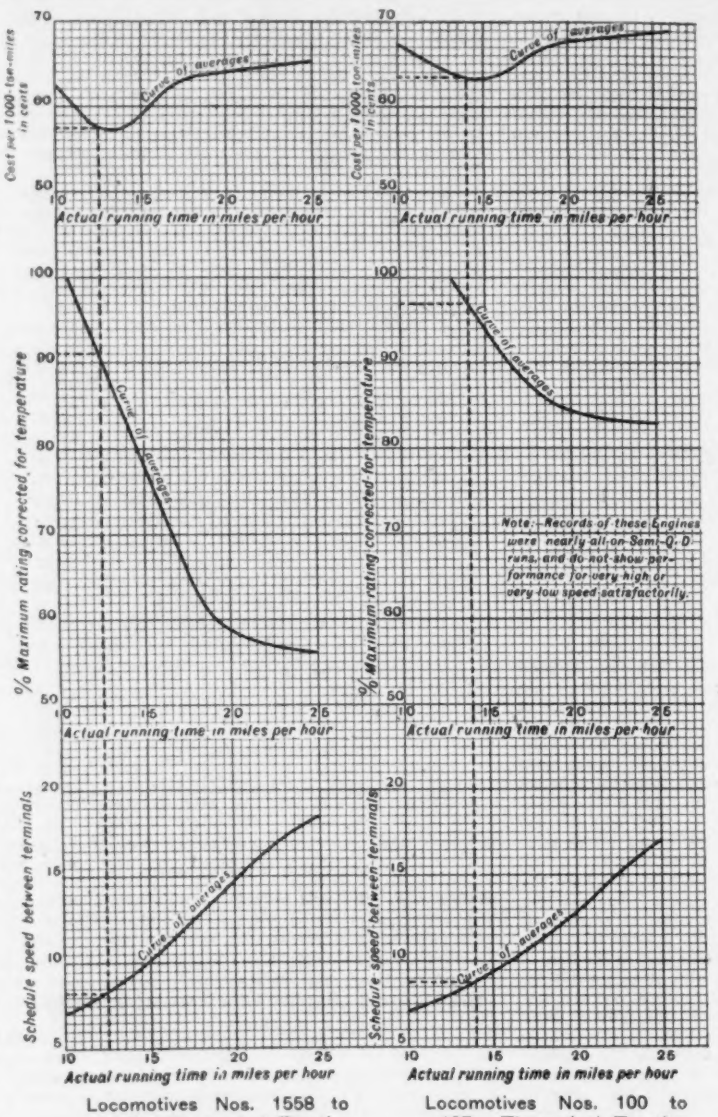
From these charts we may see graphically the comparative cost of fast and slow freight service and to a certain extent the relative economy of heavy and light motive power. When it is desired to move freight between given points on this division in a fixed time, it will be possible from these diagrams to determine the rating which must be given engines in order that they may make the wished-for schedule.

For 1,500 class compound freight engines the minimum cost is found at a speed of 13 miles per hour, which requires a rating of 88% of the maximum; but a speed of 12½ miles per hour does not increase the cost of handling appreciably and will allow 91% of maximum rating to be hauled. By referring to third chart it appears that an actual running speed of 12½ miles per hour corresponds with a schedule speed of 8¼ miles per hour between terminals.

For 100 class engines the minimum cost is shown at a speed of 15 miles per hour, which requires a rating of 94% of the maximum; but a speed of 14 miles per hour does not increase the cost of handling appreciably and will allow 97% of the maximum rating to be hauled. It also appears that running speed of 14 miles per hour corresponds to schedule speed of 8¾ miles per hour.

From the conditions existing when these records were taken, we should conclude that for the 1,500 class engines the most economical operation will be attained by trains scheduled at 8¼ miles per hour, running 12½ miles per hour and hauling 91% of maximum common rating. For 100 class engines the greatest economy of operation will be attained by trains scheduled at 8¾ miles per hour, running 14 miles per hour and hauling 97% of maximum common rating.

The records of about 500 trains given above were taken during a period of exceptionally heavy traffic and also during unusually severe weather, with the temperature in the neighborhood of zero for weeks at a time. It is probable that results in warm weather would be different. It would, therefore, be desirable to make the same determinations under summer conditions and this, it is hoped, we will be able to do later. It is to be observed that the determinations of the most economical tonnage rating by this method is independent of the rating originally assumed.


Cost per 1000 ton-miles in cents
Curve of averages
Actual running time in miles per hour
% Maximum rating corrected for temperature
Schedule speed between terminals
Note:—Records of these Engines were nearly all on Sam. Q. D runs, and do not show performance for very high or very low speed satisfactorily.

Locomotives Nos. 1558 to 1599. Theoretical Tractive Power, 36,540 lbs.

Locomotives Nos. 100 to 137. Theoretical Tractive Power, 23,314 lbs.

CHART SHOWING ECONOMICAL SPEED AND LOADING FOR FREIGHT TRAINS.

Compiled from train records of the Baltimore & Ohio Southwestern R. R.; Indianapolis Division; Jan. 1 to Feb. 15, 1904.

LOAD TEST OF A 55-FT. REINFORCED CONCRETE GIRDER, CLEVELAND, O.

An interesting example of long-span reinforced concrete girder construction is illustrated by the accompanying drawings. The girder illustrated, and another similar to it, but smaller in section, were built to carry the gallery of the New Lyric Theater, recently completed at Cleveland, Ohio. To satisfy the architects of the soundness and safety of this girder construction, the larger of the two beams was subjected to a concentrated load test on Dec. 23, 1903. The method of making this test, as well as the construction of the girder, is shown by the illustrations.

The two reinforced concrete girders span the full width of the theater, and carry the gallery floor in the manner illustrated by Fig. 1. The spans of the girders are 55 ft. in the clear. The drawing, Fig. 1, shows the cross-section of the smaller girder. The larger one is 12 ins. wide and 5 ft. deep, with an upper flange 20 ins. wide and 10 ins. deep properly reinforced to take care of buckling which might otherwise occur with so long a beam. Details of the reinforcement are shown by Fig. 2. The concrete is a 1:2:4 mixture (Vulcanite Portland cement, clean bank sand, and silica gravel broken ing used) and is reinforced with 17 longitudinal rods in top and bottom section, besides stiffening rods and about 400 stirrups of different shapes and sizes.

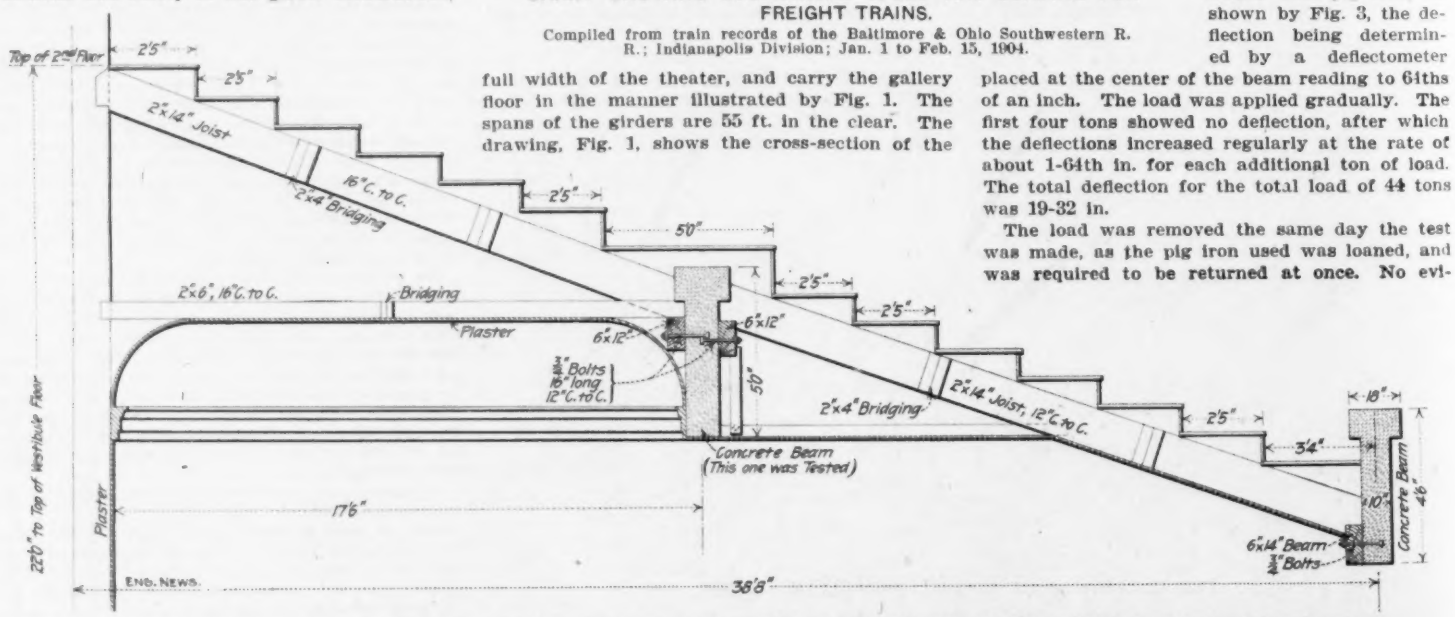


FIG. 1. SECTION THROUGH GALLERY OF NEW LYRIC THEATER, CLEVELAND, O., SHOWING REINFORCED CONCRETE SUPPORTING GIRDERS AND CONNECTIONS.

The rods used in this construction were plain round medium steel, no rods with lugs or of other special shapes being used.

The architect of the building was Mr. Frederick

The beam rests on its own concrete-steel columns and footings, so that the whole reinforced concrete construction (beam, columns, and footings) are independent of the rest of the building and would stand intact were the latter entirely removed. The building of the forms was a simple job of carpenter work. The steel for the beam was prepared and placed in the mold in two days, and the concreting was done in another day of ten hours. The centering was given a camber of 1½ ins. when erected, but during the concreting this was reduced by the load to about 1⅛ ins. The centering was removed 28 days after placing the concrete, and the beam was tested 21 days later, or seven weeks after erection.

Owing to the great weight required to test the beam, and the small surface on top to receive the load, it was decided by the contractor to place a nearly concentrated load at the center, distributed over a few feet. To accomplish this, a platform was suspended from the beam, and loaded with pig iron, as shown by Fig. 3, the deflection being determined by a deflectometer placed at the center of the beam reading to 64ths of an inch. The load was applied gradually. The first four tons showed no deflection, after which the deflections increased regularly at the rate of about 1-64th in. for each additional ton of load. The total deflection for the total load of 44 tons was 19-32 in.

The load was removed the same day the test was made, as the pig iron used was loaned, and was required to be returned at once. No evidences of any cracks or other failures were discernible.

W. Striebinger, of Cleveland. The work was executed by Mr. B. P. Faragher, of Cleveland, Ohio, to whom we are indebted for the information from which this description has been prepared.

TEST LOADS OF PILES DRIVEN WITH A STEAM HAMMER.*

By J. J. Welsh.

The lot on which this piling was done is the southeast corner of Spear and Market Sts., San Francisco, and was formerly a part of the bay which was gradually filled in, the lot therefore being "made ground."

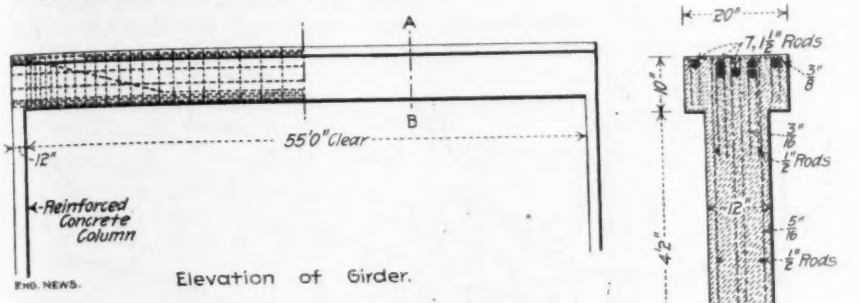


FIG. 2. DETAILS OF 55-FT. REINFORCED CONCRETE GIRDER CARRYING THEATER GALLERY.

After six piles had been driven it became evident that the ground was exceedingly soft, and inquiry among architects, engineers and pile-driving firms showed that the ground here was much more yielding than the surrounding area. This may be explained by the manner of filling in. At that time the lot was occupied by houses on stilts and the sand and rubbish were dumped around the stilts, causing the soft mud to rise almost to the same level under the houses without materially compressing it.

The new building was to be of brick and six stories in height, making necessary a careful determination of the actual resistance of the piles. The softest spot was found by driving piles in a number of places. Here a pile and follower were driven down 105 ft. without finding a hard stratum. The following day another pile was driven, which sank 1 ft. in the last two blows. On the next day it required 16 blows to sink this pile 1 ft., the first three blows having no apparent effect, while the fourth blow started it.

It was decided, as a third test, to load four piles. In order to make the conditions such as would exist under a pier 11 piles were driven into a trench. The four outer ones were left 18 ins. higher than the others, so as to bring the bearing only on these four and to give them the benefit arising from the consolidation of the material near them.

The piles were Oregon pine, 70 ft. long, and were 12 to 14 ins. at the butts and 6 to 8 ins. at the point. The test piles were spaced 4 ft. 8 ins. on centers for the short span, and 7 ft. 1 in. for the long span; on top of the four piles were set four steel plates 14 × 14 ins. × ⅞-in. thick, and on top of these were placed two 15-in. I-beams, weighing 1,000 lbs. each, bolted together to each set of piles in the long span, and upon these were placed eleven I-beams, weighing 1,000 lbs. each, which formed the platform. On this platform pig iron was piled. Before putting the platform and pig iron on, levels were taken and bench marks made. The pig iron was brought to the grounds in trucks, and each load was weighed on public scales before being delivered.

The accompanying table gives (1) the conditions and results (actual loads and settlements) for each of the four piles, (2) the calculated safe load for each by three well-known formulas, and (3) the extreme load by the Trautwine formula, which is the only one of the three giving extreme load, defined by Trautwine as the load "that will be just at the point of causing more sinking."

The three formulas are:

Sanders:* $$p = \frac{12\,w\,h}{8\,s}$$

Engineering News:† $$p = \frac{2\,w\,h}{s + c}$$

Trautwine:* $$P = \frac{51.52\,w\,\sqrt[3]{h}}{s+1}$$

where
P = the extreme load on one pile, in pounds;
p = the safe load on one pile, in pounds;
w = the weight of hammer, in pounds;
h = the fall of hammer, in feet;
s = the final penetration, in inches.

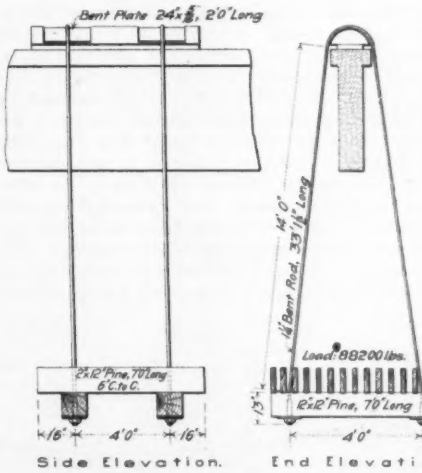


FIG. 3. DETAILS OF SADDLE AND LOADING PLATFORM FOR APPLYING LOAD TO REINFORCED CONCRETE GIRDER.

The steam hammer, used with piles Nos. 3 and 4, was known as No. 1, the heaviest made. Total weight 9,850 lbs., length 12 ft., diameter of cylinder 13½ ins., normal stroke 42 ins., weight of striking part 5,000 lbs., distance between jaws 20 ins., width of jaws 8¼ ins.

The steam hammer used is a gravity hammer, raised by steam, then automatically tripped. When the hammer reaches the pile it automatically opens up the steam, which raises it, consequently the motion is automatic, both up and down.

Now, if we figure the striking part of 5,000 lbs. as the hammer, we hardly measure the efficiency of the machine, important elements being the constant weight of 9,850 lbs. (nearly 5 tons) on the pile and the rapidity of the blows.

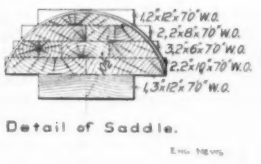
This machine could strike 65 to 85 blows per minute.

In Engineering News a comparison of the results obtained by steam and drop hammer is given. In this test a steam hammer, weighing 4,500 lbs. and having a stroke of 42 ins., was used. From 48 to 64 blows, using a follower, were required to drive the pile the last foot. In the same soil and with a pile of the same dimensions, a test was made with a drop hammer weighing 3,000 lbs. and falling 30 ft. In this case 16 blows were required, using a follower, to drive the pile the last foot. As the ratio of blows without follower to blows with follower is as one to two, the number of blows required to drive these piles the last foot without a follower would have been only one-half as many. Again, in this case, the weight of the machine, being constantly on the pile, is not taken into consideration.

We can readily understand that formulas derived from tests made under different conditions, will vary considerably, and, as the circumstances vary so greatly, it is always well to allow a large margin in calculating the strength of a pile, and a superintendent in charge of the work must depend to a considerable extent upon his own judgment rather than upon results obtained by any of the formulas, and, if any doubt exists, make an actual test.

In conclusion, the results obtained by Wilcoxon & Kearns Co., of Pensacola, from an economical point of view, will be given.

The firm was engaged in the construction of a large wharf and warehouse for the L. & N. Railroad at Pensacola, Fla., which required 7,000 piles 60 to 80 ft. long. A drop hammer of 4,200 lbs. was started on a pile which had been half driven with a steam hammer. The hammer had a drop of 60 ft. and the pile showed only 1¼ ins. penetration to each blow. The live oak cushion block was mashed into pulp. Another pile 75 ft. long was driven with the drop hammer without the hood. This took 30 min. time after it was in the leads, and required 120 blows from the top of the 75-ft. leaders. On the next pile of the same length, and 3 ft. from the one driven, they used a steam hammer and drove it to the same depth with 130 blows in 90 sec. after it was in the leaders.



This pile had no broomage, while the one along side of it, driven with the drop hammer without a hood, was broomed over 3 ft. The piles were creosoted, and cost 40 cts. per foot net, delivered on the grounds. In using the steam hammer throughout this work a saving of 21,000 ft. of piling was made, at 40 cts. per foot, or $8,400 total.

Since writing this paper the author has discovered that some of the piles had penetrated through old piles which had been covered up since the filling in. This accounts for the fact that these particular piles could not be driven with the drop hammer, while the steam hammer sent the piles clean through the old piles to the proper depth.

*A paper read before the Technical Society of the Pacific Coast May 27, 1904, and published in Journal Assoc. Eng. Soc. September, 1904.

*Trautwine's Civil Engineer's Pocket Book, Editions of 1902 and 1904, page 592, where the Trautwine formula is given in the form: $P = 2{,}240\,\frac{0.023\,w\,\sqrt[3]{h}}{s+1}$

†Engineering News, Dec. 29, 1888, p. 511. For drop hammer, c = 1. For steam hammer, c = 0.1. See Engineering News, Nov. 17, 1892, p. 470.

[The factor of safety used in the Engineering News formula is 6.—Ed.]

RESULTS OF TEST LOADINGS OF PILES COMPARED WITH FORMULAS FOR BEARING POWER.

Pile Number.	Type of Hammer.	Number of Blows Struck.	Penetrations* and Falls.	Time of Driving, Minutes.	w, Weight of Hammer, Pounds.	h, Final Fall, Feet.	*p h	s, Final Penetration, † Inches.	Safe load, p, in pounds. Sanders.	Engineering News. Drop.	Engineering News. Steam.	Trautwine.‡	First Day's load, 80 tons; Total, 44,800 lbs. per Pile. Settlement, Inches.	Second Day's load, ** 144 tons; Total, 80,640 lbs. per Pile. Settlement, Inches.	Extreme Load, P, by Trautwine's Formula.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1	Drop	117	24 ins. per blow of 8 ft.	15	3,080	12	2.289	6	9,240	10,560	.....	12,958	½	½	51,932
2	"	92	72 ins. first blow, 5 ft. 18 ins. per blow; 9 blows, 5 ft.	3	3,080	29	3.072	9	14,887	17,864	.....	12,187	½	¾	48,748
3	Steam	62	513 ins. first blow. 24 ins. per blow, 3 blows	2	5,000	3.5	1.518	6	4,375	.....	5,738	13,966	⅜	⅜	55,864
4	"	120	429 ins. first blow.	3	5,000	3.5	1.518	4	6,562	.....	8,536	19,552	⅜	¾	78,208

* See also column 9. † See also column 4.

‡ Trautwine says: "As to the proper load for safety, we think that not more than one-half of the extreme load given by our rule should be taken for piles *thoroughly* driven in *firm* soils; nor more than one-sixth when in river mud or marsh; assuming, as we have hitherto done, that their feet do not rest upon rock. If liable to tremors, take only *half* these loads." We here take safe load, p = ¼ extreme load, P.

** Left on for a little over two weeks. No further settlement.

THE PHILADELPHIA FILTRATION SYSTEM.*

(Torresdale Tunnel Conduit.)

In planning the new $26,000,000 filtered water supply for the city of Philadelphia, it was decided to locate the largest of the four filter plants on the Delaware River below Torresdale, and to take the filtered water from this plant by means of an inverted siphon to Lardner's Point, where it would be pumped into the distribution system. The works as planned consisted, therefore, of three parts, the filter plant, the inverted siphon and the pumping station. In the following paragraphs it is proposed to describe only the siphon in detail.

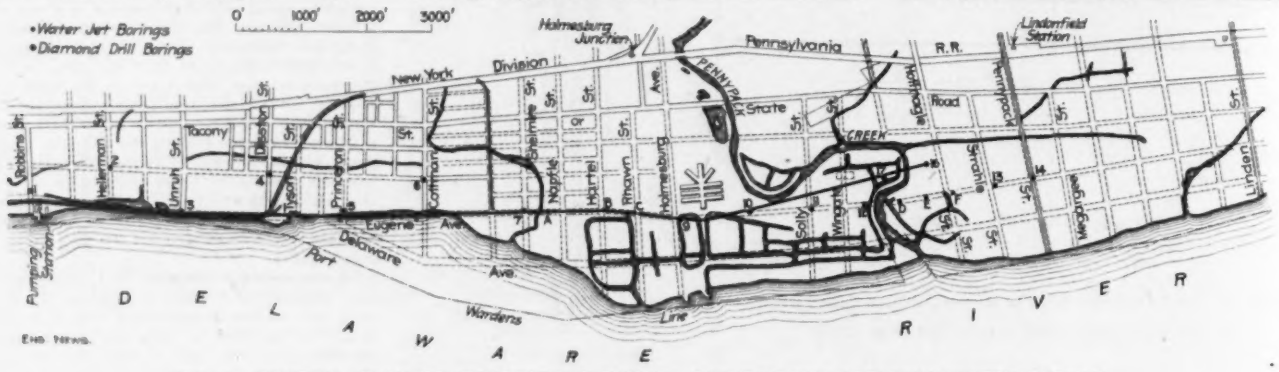


FIG. 1. GENERAL PLAN OF TORRESDALE TUNNEL FOR THE PHILADELPHIA WATER SUPPLY.

This is a 10½-ft. tunnel 13,809 ft. long, with a vertical shaft at each end, one shaft connecting with the clear water reservoir at Torresdale and the other with the pump well at Lardner's Point.

GENERAL DESCRIPTION.—Beginning at the Torresdale end, the conduit consists of a vertical shaft 10½ ft. in diameter, which connects by a vertical curve with a circular tunnel also 10½ ft. in diameter. At the opposite end of this tunnel a vertical curve connects it with a vertical shaft which is 10½ ft. in diameter for part of its height and then funnels out to a diameter of 21 ft. The shaft at Torresdale is about 125 ft. deep and that at Lardner's Point is about 105 ft. deep; the exact length of the tunnel between centers of shaft is 13,809.18 ft., shaft No. 11 being shifted slightly to the north.

The alinement and location of the tunnel are shown by Fig. 1. It will be noted that the alinement is substantially parallel to the river, the tunnel being nowhere far from the river bank and in one place being under water. In profile the tunnel has an upgrade of 9 ins. in 1,000 ft. from Torresdale to Lardner's Point, the elevations at bottom of invert at these two points being respectively 99.41 and 89.07, the tunnel having been sunk 10 ft. lower than the elevations shown on the plan, due to encountering treacherous material in shaft No. 9. Fig. 2 is a profile of the tunnel showing the character of the ground and the location of the various shafts, diamond drill borings, etc. The tunnel and the shafts are lined with brick masonry except as noted farther on; the backing is of concrete throughout. This lining will be finished smooth to reduce friction. The carrying capacity of the tunnel is figured at 320,000,000 gallons per 24 hours under normal conditions, and it is expected that this can be increased to 350,000,000 gallons by increasing the speed of pumping and slightly lowering the level in the pump well at Lardner's Point.

MATERIALS PENETRATED.—The diameter of the geological strata along the line of the tunnel as determined by the borings is shown by the profile Fig. 2. The borings were spaced 500 ft. apart, but were staggered in plan, it not being decided at the time just on what route the tunnel would be located, so that they occurred only every 1,000 ft. on the actual tunnel line. In general the ground along the line of the tunnel is soft material—mud, sand, gravel and clay—overlying bedrock. The tunnel is entirely in rock, but the various working and permanent shafts had, of course, to penetrate the soft material. The greater part, about 86%, of the rock pierced by the tunnel is a hard biotite gneiss, but there are considerable amounts of schistic and micacious rock, and in places this softer rock was disintegrated to about the consistency of mud. In some places the biotite gneiss was comparatively free from water and gave a dry excavation, but the softer rock carried large quantities of water. This inflow was sometimes concentrated in streams, but it often exuded in small drops over a rock surface of considerable area. The latter circumstance made the task of confining the water, particularly when it came in a shower from the roof, one of some difficulty.

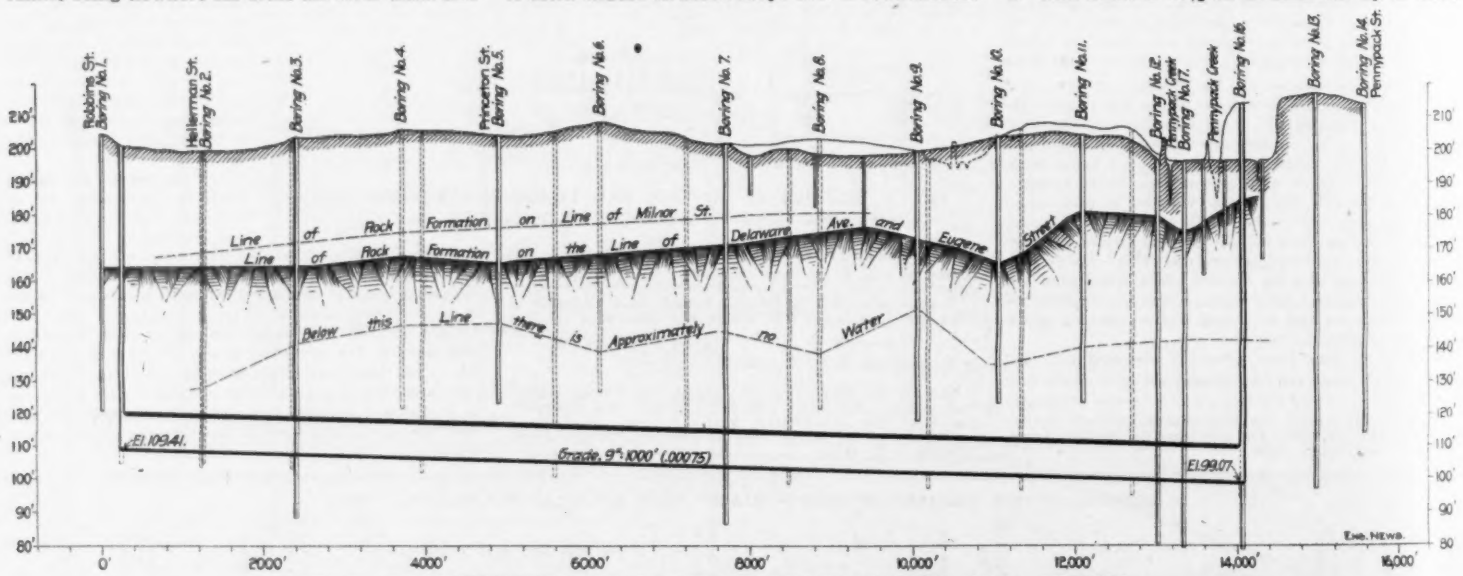

Robbins St. Boring No. 1.
Hellerman St. Boring No. 2.
Boring No. 3.
Boring No. 4.
Princeton St. Boring No. 5.
Boring No. 6.
Boring No. 7.
Boring No. 8.
Boring No. 9.
Boring No. 10.
Boring No. 11.
Boring No. 12.
Pennypack Creek Boring No. 17.
Pennypack Creek
Boring No. 16.
Boring No. 13.
Boring No. 14. Pennypack St.
Line of Rock Formation on Line of Milnor St.
Line of Rock Formation on the Line of Delaware Ave. and Eugene Street
Below this Line there is Approximately no Water
El. 109.41.
Grade, 9" in 1000' (.00075)
El. 99.07.
Eng. News.
0' 2000' 4000' 6000' 8000' 10,000' 12,000' 14,000' 16,000'

FIG. 2. PROFILE OF TORRESDALE TUNNEL, SHOWING DIAMOND DRILL BORINGS.

SHAFTS.

The engineers' plans called for the construction of the tunnel from eleven shafts. Two of these were to be permanent shafts and nine were to be working shafts, which were to be closed and filled on completion of the tunnel. These shafts were to be spaced from 1,000 ft. to 1,700 ft. apart, as shown by the profile, Fig. 2. The adoption of so many shafts was for the purpose of hastening construction, it being deemed of the utmost importance to have the plant in operation at the earliest possible date.

PERMANENT SHAFTS.—The two permanent shafts forming a part of the siphon are shaft No. 1 and No. 11, the former being located at the Torresdale end. These shafts are alike in their general construction, but differ in their dimensions. Each consists of a brick-lined steel cribbing through the soft ground and into the bedrock for a part of the depth, and of a masonry lined rock shaft for the remainder of their depth. The construction of the two shafts is shown by Figs. 3 and 4.

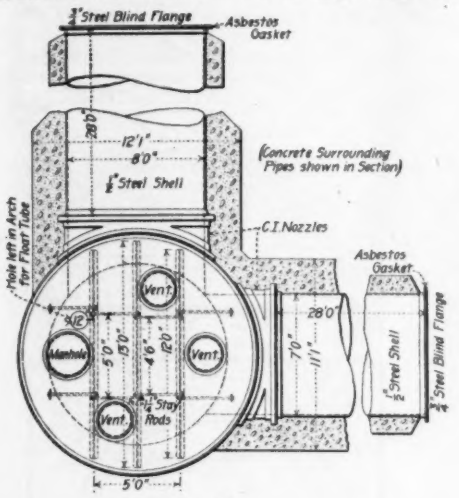
Fig. 3 shows the details of shaft No. 1. This shaft is 10½ ft. inside diameter throughout. The upper part, as will be seen, consists of a sheet steel cylinder having a cast-iron shoe or cutting edge at its bottom and lined with 18 ins. of brickwork. The steel cylinder penetrates the soft material and far enough into bedrock to be sealed against the inflow of water from the soft material above the rock. Fig. 5 is a detail of the cast-iron cutting edge; the other details of the cylinder and those of the top flooring, vents, manhole, ladder and platforms are clearly shown by the general drawings of Fig. 3. The curbing of the shaft is pierced by an inlet pipe from the clear-water reservoir, and a second inlet is provided for when the occasion demands it.

The details of shaft No. 11 are shown by Fig. 4. This shaft is 10½ ft. in diameter up to elevation + 164.5, and then funnels out to a diameter of 21 ft., which continues to the top. The steel shell which is lined with brick to form the upper curbing of the shaft is 24½ ft. in outer diameter, and its bottom is a cast-iron cutting edge of the same construction as is used in shaft No. 1. The other details of the shaft construction are fully shown by the drawings. The outlets of the shaft are two in number, the longer of which leads to the pump well.

WORKING SHAFTS.—Shafts No. 2 to No. 10

*A general description of the Philadelphia filtration system was published in Engineering News of Aug. 18 last.

inclusive, were excavated for construction purposes. They were constructed like the permanent shafts by sinking steel cylinders through the soft material and sealing them into the rock, the cylinders being lined with brickwork. No lining was used elsewhere in the shaft except where necessary to sustain loose rock or to cut off large leaks. The



Top Plan.

working shafts were made about 10½ ft. in diameter, and each was provided with the necessary framework for an elevator cage. At the bottom of each shaft a sump about 5 ft. deep was cut in the rock to serve as a pump well for removing the drainage water from the adjacent workings. The section, Fig. 6, shows a typical working shaft. After serving their purpose in the construction of the tunnel, the working shafts were filled flush with the ground surface. To carry this filling the tunnel lining was locally reinforced and two relieving arches were sprung across each shaft, as shown by Fig. 6. The filling was required to be done with selected material deposited in 6-in. layers and rammed.

TUNNEL.

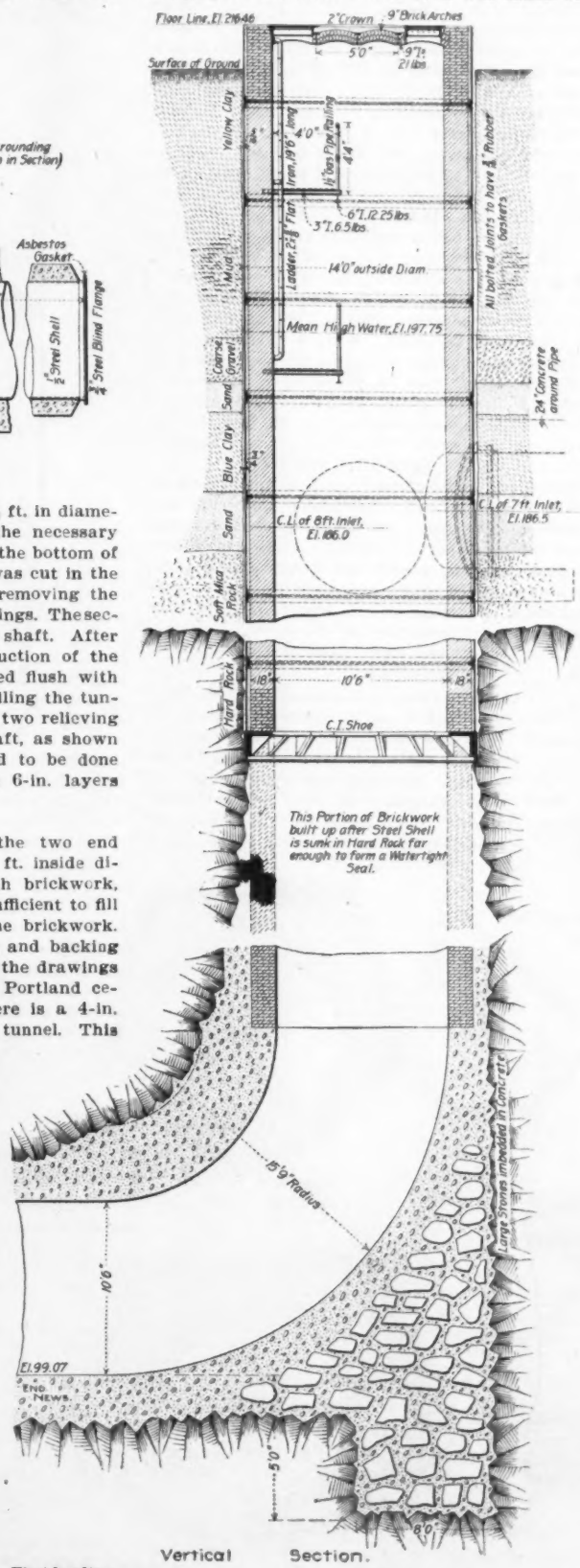
The tunnel conduit connecting the two end shafts is 13809.18 ft. long and 10½ ft. inside diameter. It is lined throughout with brickwork, backed by a thickness of concrete sufficient to fill completely all the voids back of the brickwork. The standard sections of brickwork and backing for different materials are shown by the drawings of Fig. 7. All brickwork is laid in Portland cement mortar. Under the invert there is a 4-in. tile subdrain the full length of the tunnel. This drain is filled around with concrete and all sumps at the feet of the working shafts are concrete filled. Where timbering is left in the work it is carefully packed around with the concrete backing and afterwards the remaining voids are grouted.

METHODS OF CONSTRUCTION.

SHAFTS.—The first work done was the sinking of the working shafts, and excavation began on all of these in December, 1901. At shaft No. 4, an explosion of a charge of dynamite caused such an accident to the steel shell which had been partially sunk, that it could not be recovered; a second shaft was accordingly started at this point in April, 1902. The dates of beginning the several shafts, their depths and the date of turning the first heading are given in the following tabular statement:

Shaft.	Depth, ft.	Commenced.	First heading.
1	129	April 16, 1902	Nov. 6, 1902
2	121	Dec. 17, 1901	July 18, 1902
3	121	Nov. 26, 1901	April 2, 1902
4	111	March 19, 1902	July 17, 1902
5	113	Dec. 11, 1901	April 2, 1902
6	109	Dec. 4, 1901	April 4, 1902
7	107	Dec. 25, 1901	Aug. 12, 1902
8	109	Dec. 6, 1901	April 5, 1902
9	111	Dec. 14, 1901	April 11, 1902
10	106	Dec. 4, 1901	April 3, 1902
11	109	March 3, 1902	Oct. 10, 1902

The method of driving the shafts was to sink a steel cylinder through the soft material and into the rock, where it was sealed against the entrance of water from the material above the cutting edge. These cylinders were sunk, except in two cases where compressed air was used, by means of open dredging, using for the most part orange-peel excavators. The rock was taken out



Vertical Section.

Fig. 3. General Details of Shaft No. 1, Torresdale Tunnel.

by blasting in the usual manner. The sinking of the shells was troublesome in a few cases owing to the tipping of the shell. After sinking each shaft was equipped with an elevator cage and the usual other apparatus. While the contractors were left largely to their own devices in sinking the working shafts, they were compelled to obey certain restrictions in sinking the permanent shafts Nos. 1 and 11. It was essential that the curbing and lining of these shafts should be truly vertical and uniformly cylindrical. To ensure the vertical sinking of the cylinder, use was made of the guiding framework shown by the

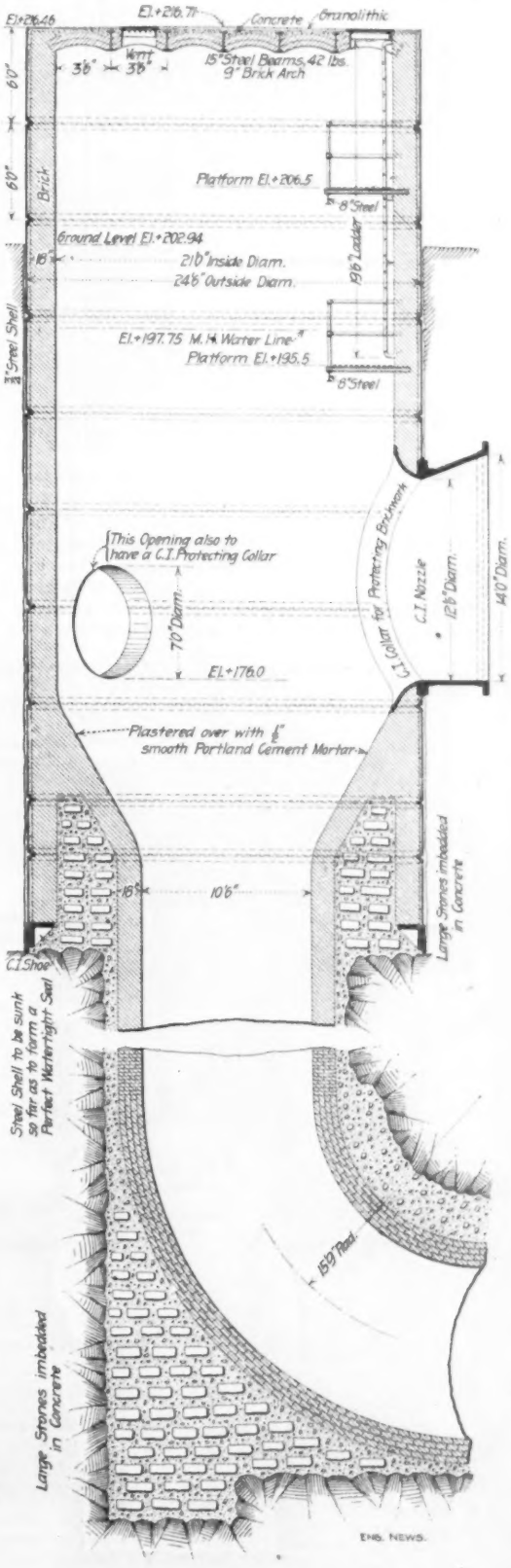


Fig. 4. General Details of Shaft No. 11, Torresdale Tunnel.

drawings of Fig. 8. These drawings explain themselves and need no further comment, except to state that they show the construction for shaft No. 11 and that this construction was slightly modified by the smaller dimensions of shaft No. 1.

TUNNEL.—The tunnel is being excavated by the

single wide heading and bench method, work being carried in both directions from each intermediate shaft, and in one direction from the end shafts. The relative portions of the section taken out in heading and in bench are indicated by Fig.

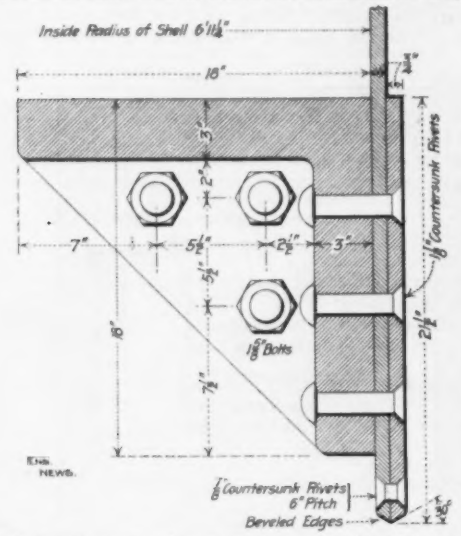


Fig. 5. Cross-Section of Cast-Iron Cutting Edge for Shaft Curbing.

9. This drawing and the companion ones of Fig. 10 and Fig. 11 show the method of taking out the section. A wedge shaped center cut is made by means of the holes A. This cut is then enlarged by blasting the relief holes B. The side holes C and the roof holes D and E are then blasted, generally in one round. The blasting of these sets of holes open up the heading. The bench is then taken out by blasting the holes F.

Compressed air percussion drills are used for all the drilling. These are mounted on shafts for the heading work and on tripods for the bench work. The view, Fig. 12, shows the arrangement of the drill mountings in one of the headings. The same feature is also indicated by Figs. 10 and 11. The view, Fig. 12, shows two drills mounted on each column, but in many of the headings two and three drills were worked by shifting them on the columns as indicated by Figs. 10 and 11. Details of the labor force and time account are available for the excavation from 15 headings. These have been furnished to us by Mr. H. B. Hanger, of the firm of Mason, Hoge & Co., of Frankfort, Ky., who are the contractors for Sections 1, 2 and 3, comprising shafts 1 to 8, inclusive. These contractors employed two drills in eleven of the headings worked by them, and three drills in the other four headings. As a rule 21 holes were drilled for the heading cut and four holes were drilled for the bench cut. The heading holes were drilled 8 ft. deep and the lift of rock removed by each blast was about 5½ ft. The charge was 4 lbs. for each hole. The heading holes were fired in the order already noted. A blast was made at least every 20 hours. From 4 to 5 hours were required in mucking out and setting up the drills for each blast, and 13 hours were occupied in drilling. The mucking proceeded continuously; seven muckers were employed in the heading and seven on the bench. These were worked in two ten-hour shifts.

The most treacherous rock was found between shafts 8 and 11 and considerable timbering was required. The character of this timbering varied with the conditions. At points where heavy falls of rock in large pieces followed the blasts the various styles of timbering shown by Fig. 13 were employed. At other points where the roof rock was more fragmentary in character, a rather close and tight segmental roof timbering was adopted.

The lining construction calls for brief description only. The first operation was the construction of the concrete saddle for the lower half of the brick lining. Mounted centers were set up and fastened at the proper height above the floor of the excavation. These centers were then lagged in convenient sections and the concrete filled behind the lagging. The lagging was begun at the bottom and was placed a piece or two at a time and blocked up against the centers. The concreting then proceeded until additional lagging pieces were needed. Fig. 14 shows a section of tunnel with the centers and lagging for the saddle in place. After the concrete had set and the lagging was removed the upper surface of the saddle was plastered with a sufficient coating of cement mortar to fill all roughness and give a smooth bearing to the invert brickwork. This was then built up in the usual manner. The construction of the roof arch then followed. The view, Fig. 15, shows the centering of this work, and, as will be observed, this view happens to be in a portion of the tunnel where the rock required heavy timbering. The brickwork of the roof arch was constructed in the regular way, and about all that needs to be noted is that the concrete back-

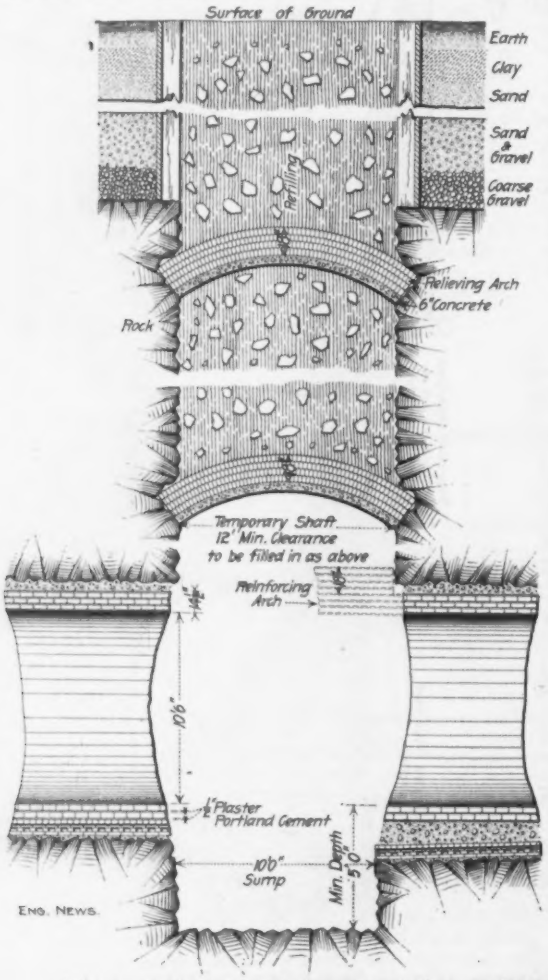


FIG. 6. GENERAL DETAILS OF TYPICAL WORKING SHAFT, TORRESDALE TUNNEL.

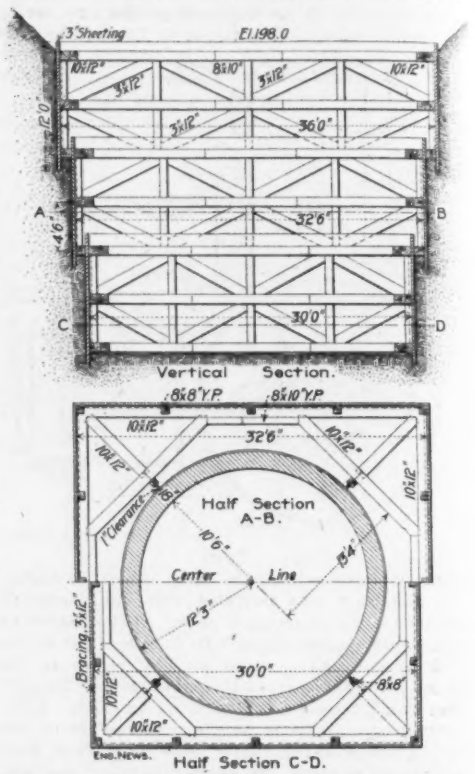


Fig. 8. Frame for Guiding the Sinking of Shaft Cylinders, Torresdale Tunnel.

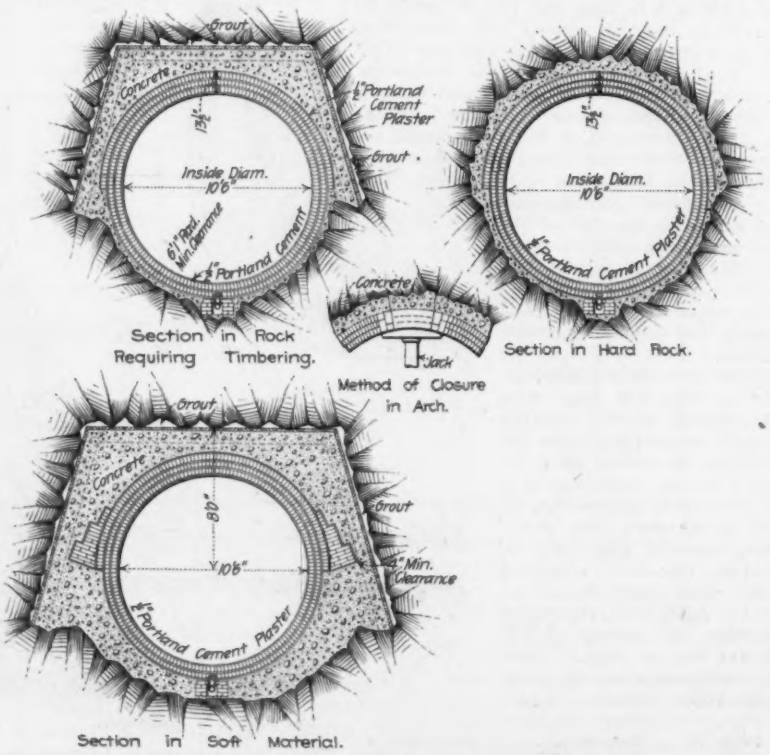


FIG. 7. STANDARD SECTIONS OF TORRESDALE TUNNEL.

ing or filling proceeded with the progress of the brickwork. This provision was strictly insisted upon by the engineers.

MATERIALS AND WORKMANSHIP.

The principal materials used in constructing the tunnel and shafts were concrete, brick and cast-iron and steel tubing.

CONCRETE.—The concrete employed was composed of 1 part Portland cement, 3 parts sand and 5 parts broken stone or gravel. As a matter of fact, very little gravel concrete was used. The specifications for the cement were rather strict, and we quote them as follows:

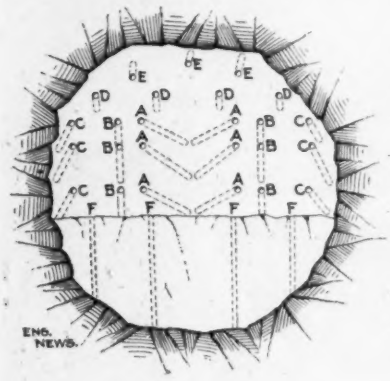


Fig. 9. End View of Heading Showing Drill Holes.

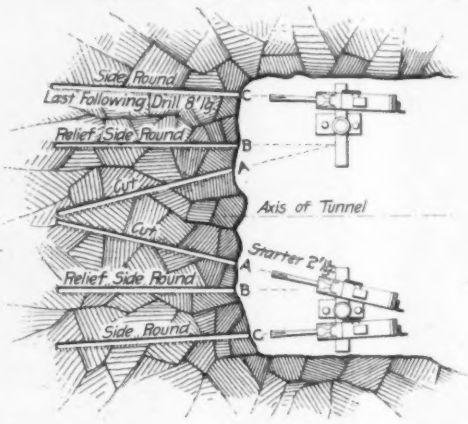


Fig. 10. Plan of Heading Showing Drill Holes.

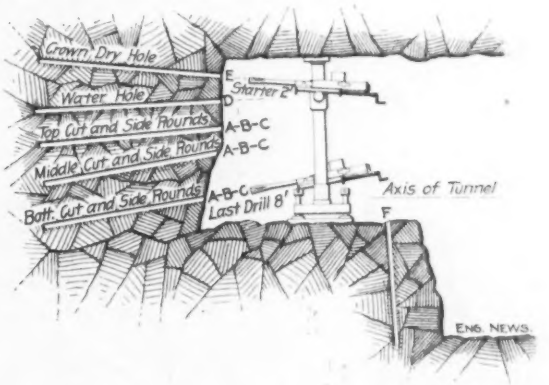


Fig. 11. Longitudinal Section of Heading Showing Drill Holes.

Portland cement shall have a specific gravity of not less than 3.1, and shall leave by weight a residue of not more than 1% on a No. 50 sieve, 10% on a No. 100 sieve, and 25% on a No. 200 sieve. The sieves shall be made of brass wire cloth, having approximately 2,400, 10,200 and 35,700 meshes per square inch, respectively, the diameters of the wires being respectively 0.009, 0.0045 and 0.002 in. Pats of neat cement ½-in. thick with thin edges after hard set when immersed in water maintained at a temperature of 175° F., shall remain firm and hard, and show no signs of swelling, "checking" or disintegration. The cement shall contain not more than 1.75% of anhydrous sulphuric acid (SO_3). It shall require at least 20 minutes to develop the "initial" set, this being determined by means of the Vicat needle from prts of neat cement of normal consistency, the temperature being between 60° and 70° F. Briquettes of 1 sq. in. in cross-section shall develop the following ultimate tensile strengths:

Age.	Strength, lbs.
24 hours (in water after "hard" set), net cement....	175
7 days (1 day in air, 6 days in water), neat cement..	500
28 days (1 day in air, 27 days in water), neat cement.	600
7 days (1 day in air, 6 days in water), 1 cement to 3 standard quartz sand.....	170
28 days (1 day in air, 27 days in water), 1 cement to 3 standard quartz sand....	240

The sand was specified as is usual to be clean and sharp and graded in size. All parts larger than ¼-in. were required to be screened out, and any sand containing more than 3% by weight of loam, clay, dirt or other impurities was forbidden. For broken stone granite, trap or other hard rock was specified. All pieces less than ¼-in. in size and all larger than 1½ ins. were required to be screened out. The material between these sizes was required to be "graded from fine to coarse." Either machine-mixing in cubical mixers or hand-mixing was allowed. The deposition of the concrete was required to be in 6-in. rammed layers. Exposed surfaces were to be kept wet during setting, and junctions of old and new concrete were required to be made by washing the older surface and, if requested, by slushing it with neat cement grout.

BRICK MASONRY.—Only high-grade bricks were allowed, and they were required to be laid in a 1 Portland cement, 3 sand mortar. Each brick was required to be wetted before laying, and old brickwork was required to be cleaned and wetted before laying new. Connections were required to be made by leaving the old work racked back, toothing not being allowed. The specification for the brickwork of the lining was as follows:

The bricks shall be laid with their lengths parallel to the axis of the tunnel; each longitudinal line of brick breaking joints with the adjoining line. No headers shall be used in the work. The bricks with the smoothest edges shall be selected to form the inside ring. The collar joints between the rings shall not be less than ⅜-in. nor more than ½-in. The last five courses of brick at the crown of the arch shall be laid with dry tight joints and then grouted with neat cement grout.

The specifications for cast iron, steel, stone-work, etc., were those employed generally for first-class work of similar character, and they will not be given.

ENGINEERS AND CONTRACTORS.

The construction of the Torresdale tunnel has been prosecuted under the direction of the following engineers and contractors: Chief Engineer in charge of the work appurtenant to the improvement of the water supply, John W. Hill, M. Am. Soc. C. E.. Contract No. 14, Torresdale Conduit, First Assistant Engineer in charge to March 1, 1902, Richard I. D. Ashbridge, Assoc. M. Am. Soc. C. E.; since March 1, 1902, T. Nelson Spencer, Assoc M. Am. Soc. C. E. Second Assistant Engineers, Charles P. Bower to May 10, 1903; S. M. Purdy to July 1, 1903; Moriz Bernstein since July 16, 1903; William H. Thorpe since May 1, 1903. Third Assistant Engineers, Frank McHugh since July 23, 1903; John D. Quinn since May 20, 1903. Mining Inspectors, A. York Smith to October, 1902; Geo. Hein since Oct. 18, 1902; John E. Peters, head masonry inspector. Contractor, Mr. Daniel J. McNichol, Philadelphia, Pa. Subcontractors, Section No. 1, Shafts Nos. 1, 2, 3 and 4; Section No. 2, Shafts Nos. 5 and 6, and Section No. 3, Shafts Nos. 7 and 8, Mason, Hoge & Co., Frankfort, Ky. Section No. 4, Shafts Nos, 9, 10 and 11, managed by Contractor, D. J. McNichol, under the supervision of J. Clifford Fitch, to March 2, 1903, and since March 2, 1903, by L. G. Burleigh.

The total cost of the tunnel will be about $1,350,000.

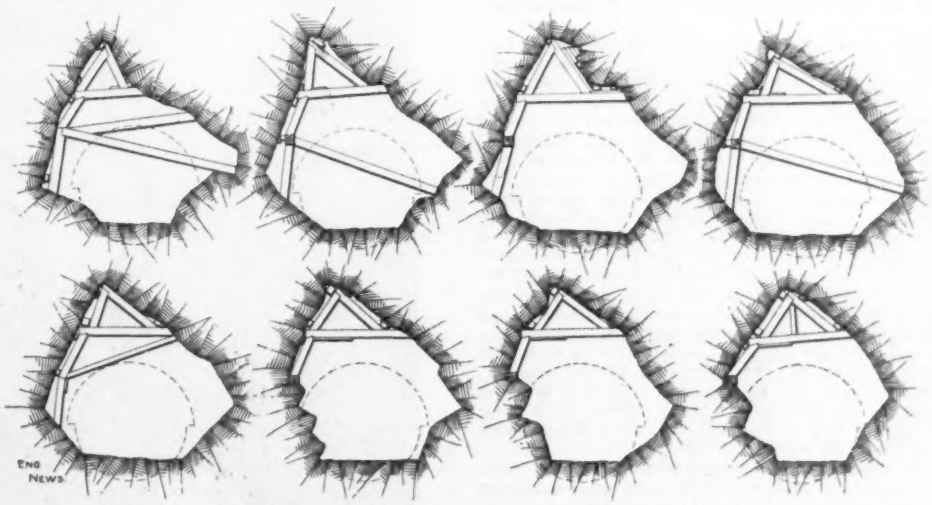


FIG. 13. SECTIONS SHOWING TIMBERING OF LARGE ROCK FALLS.

OILED ROADBEDS OF RAILWAYS IN CALIFORNIA.*

The Atchison, Topeka & Santa Fe Ry. Co. has in California 624 miles of oiled roadbed, the mileage of such roadbed on the divisions being as follows: Valley division, San Francisco to Bakersfield, 204 miles; Arizona division, Mojave to Needles, 248 miles; Los Angeles division, Barstow to National City, including local branches, 172 miles. The first work of oiling was undertaken on the Los Angeles division in the year 1898, since which time the Arizona and Valley divisions have been similarly treated. Only unballasted track was oiled, there being little or no dust in connection with ballasted track. All of the above mentioned track has been oiled several times, and the majority of it was about Oct. 1, this year, given an additional application. The work is done by a special train consisting of a locomotive and five or six ordinary tank cars loaded with oil, to the end of the last car of which is attached a sprinkling apparatus, consisting of an ordinary wrought-iron perforated pipe 18 ft. in length, occupying a position at right angles with the track. Each oil car is connected with the other so that all drain into the rear car. For track sprinkling purposes the oil produced in the Kern River fields is used, principally because it has a heavier base than the product obtained in Southern California, which makes it better adapted for the purpose. When the oiling is done in the summer, and the weather is warm, it is not necessary to heat the oil, but if the work is performed in the early spring or fall, it is necessary to heat it, which is done by means of steam from the locomotive, each tank car being equipped with heater coils for that purpose. A strip 18 ft. in width is oiled, using on the first oiling about 70 barrels per mile, and from 50 to 60 barrels on subsequent oilings, depending largely on the nature of the soil and how much the previous oiling has been disturbed. The cost of the oil varies, being regulated by the market. The cost of oiling has averaged about $14 per mile, which amount includes work-train service, wages of train crew, section labor, cost of oil, etc.; in addition to which the American Dustless Roadbed Co., of Camden, N. J., is paid a royalty per mile for the privilege of oiling, they holding a patent on the process. Thus far excellent results have been secured from the oiling of the roadbed, not only in the matter of laying the dust and arresting the growth of weeds, but also in holding together the material constituting the subgrade, especially in regions where the soil is very light. Experience has demonstrated that in very dusty countries the roadbed should be oiled about twice each year—once in the early spring after the track has been put up for the summer, and again in the fall just before the cold weather sets in. It has been found that one application will not kill the weeds, although it has a discouraging tendency. With a newly-oiled roadbed it requires at least three seasons before the weeds are effectually destroyed. The action in the matter is considered merely mechanical, in that the oil does not destroy vegetable life, but renders the soil unsuitable for its support after it has once be-

*Abstracted from Bulletin No. 2, Department of Highways of the State of California, Sacramento.

come thoroughly saturated; in other words, while the weeds will readily sprout and force their way up through a thin overlying layer of oiled ground, after the ground is once completely saturated it furnishes no further support for the plant, but so long as the crust is thin enough to allow it to force its way through, it will continue to grow. There is about one-half inch penetration for each oiling, and it follows that if the roadbed is disturbed, either by tamping or putting in new ties, the dust comes to the surface and re-oiling is necessary. For that reason it is desirable, especially under conditions existing in California, to put on a heavy force in the spring, as soon as the ground is dry enough to work, to put the track up in line and surface, then oil it and leave it untouched as late into the summer as is possible, in order to obtain the maximum efficiency of the oiling. The fall application serves not so much to keep down the dust as to assist in drainage, as the oiled crust will shed water off almost any kind of material. Another benefit from an oiled roadbed is the wear on machinery which it saves, and a reduction in the number of hot boxes, the value of which is considerable.

The Southern Pacific Co., Pacific System, has in California 166.41 miles of oiled roadbed, the mileage by counties being as follows: Alameda, 31.76; Contra Costa, 5.49; Fresno, 1.28; Los Angeles, 22.82; Monterey, 18.20; Napa, 2.05; Riverside, 14; Sacramento, 58; San Bernardino, 32.30; San Joaquin, 8.88; Santa Barbara, 2.70; Santa Clara, 20.38; Solano, 3.40; Sonoma, .87; Tulare, .50; Yolo, 1.23. The use of oil, which was begun in October, 1899, has been confined to those portions of the line on which there is no ballast, there being little or no trouble from dust on the ballasted portions. The oil is distributed on the roadbed from an oil car to which is attached a sprinkler, a strip 14 ft. wide being oiled. About 100 barrels per mile have been used on the first application, and a somewhat less quantity has generally been used once a year for the subsequent applications. Experience indicates that oil should be applied every six months to get the best results. When sprinkling is to be done during cold weather the oil is heated in the tank by steam coils, but heating is avoided when possible. Kern River oil is used. The average cost of oiling is $15 per mile, exclusive of the cost of the oil.

The system of the Pacific Electric Ry. Co., with headquarters in Los Angeles, has 140 miles of oiled roadbed (single track), most of which is in Los Angeles County. The work of oiling was begun in 1902. The lines are constructed on various kinds of soil—sand, adobe, hardpan, clay, gravel, black earth, and earth containing alkali. Before oiling, the roadbed is dressed to the proper cross-section for drainage, and the surface is scratched with a rake so that the oil will penetrate a short distance below the surface. The roadbed is not touched after applying the oil, except for repair work. The oil is applied by an apparatus consisting of a sprinkler made by drilling a row of holes in a 4-in. pipe, which is connected by a "T" with an oil car. The roadbed is oiled between the rails, and for a distance of 3 ft. outside the rails; on city streets for a distance of 2 ft. outside the rails. On the first application, from 65 to 90 barrels of oil are used, according to the nature of the material composing the roadbed. After an interval of three to six months a second application of oil is made, just enough to form a crust which will hold down the dust in summer and shed ordinary rains in winter. The oil is heated to 120° to each car of oil. With everything in working order, a roadbed can be oiled at a rate of 3 to 4 miles an hour. When a permanent surface or crust has been formed, the only renewals needed are those caused by disturbances to the roadbed due to the repair of the track. Sand or gravel oiled roadbeds are easiest to keep in good condition. In poorer material, where the tracks have to be resurfaced often, the oiled material soon disappears, although before repair work is begun, the crust or oiled material is carefully removed, and is replaced when the repairs are completed. The results of oiling the roadbeds are regarded very favorably by the company. The oiled surface protects both passengers and car machinery from dust in the summer, and sheds an ordinary quantity of water from the roadbed in the winter. And where tracks are constructed along the sands of the seashore the oiled sand near the rails is so heavy that it is not blown away by the sea breezes, which otherwise whip the sand from under one side of the track and pile it up on the other. The company will oil 110 miles more of roadbed during 1904.

The Oakland Traction Consolidated Co. has in Oakland and Berkeley an aggregate of 57 miles of oiled roadbed extending 2 ft. each side of the rails.

The United Railroads of San Francisco has oiled one mile of double-track electric railway roadbed between Burlingame and San Mateo, in San Mateo County. The first application, of 125 barrels, was made July 6, 1904. Kern County fuel oil of 16½° gravity being used. One week later the same quantity was again applied, the treatment continuing until an aggregate of 300 barrels of oil had been used. The dust and loose sand were removed before each application, and replaced after the ground had become well saturated with oil. This caused the formation of a thick, gummy substance, similar to bitumen. All the oil was applied cold, with a sprinkling tank electric car, the holes in the sprinkler being 5-16-in. in diameter and 1 in. between centers. The results of the oiling have been highly satisfactory, the main object being to prevent dust.

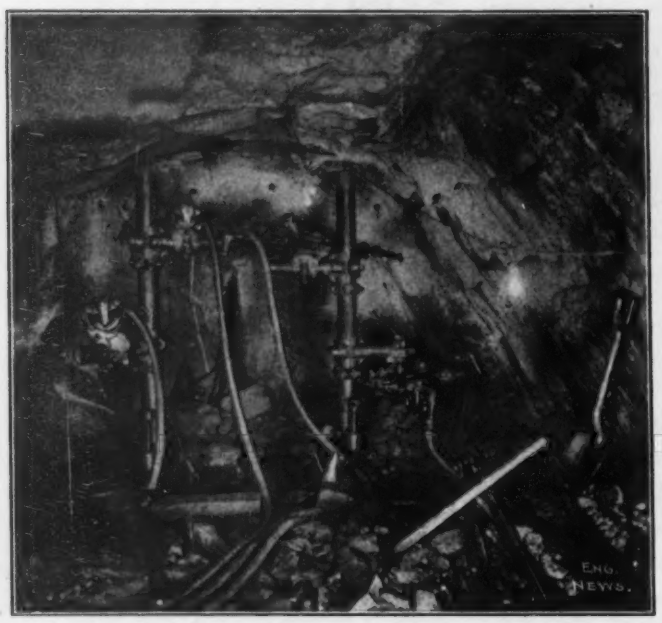


FIG. 12. VIEW INSIDE HEADING SHOWING DRILLS AT WORK ON HEADING CUT.

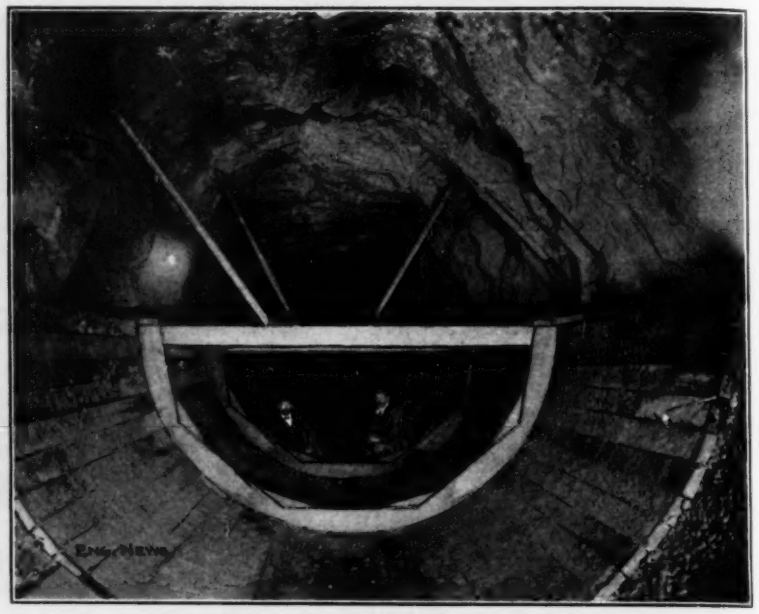


FIG. 14. VIEW INSIDE TUNNEL SHOWING FORMS FOR CONCRETE SADDLE FOR LINING.

FIG. 15. VIEW INSIDE TUNNEL SHOWING CENTERS AND STAGING FOR CONSTRUCTING ROOF ARCH.

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Tunnels for water supply seem predestined to charges of fraud and maladministration in their construction. Just now the citizens of Philadelphia are in a state of ferment over leaks in the new three-mile conduit, known as the Torresdale Tunnel, which has recently been completed for the filtered water supply of that city. It does not appear certain how extensive these leaks are, nor is the cause to which they are due definitely settled. The newspaper accounts of the trouble are so contradictory as to be almost worthless as evidence. Practically the only facts that can be sifted out of the mass of accusations and denials are that the leakage amounts to between 1,500,000 and 2,000,000 gallons per 24 hours, and that the greater bulk of this water gains entrance to the tunnel at the bottoms of shafts 1 and 2 where the sumps providing for drainage during construction were improperly sealed when work was finished. With these large leaks closed, it is estimated by Mr. John W. Ledoux, M. Am. Soc. C. E., who personally examined the work and who stated his opinion to the reporter of the Philadelphia "Item" that the total leakage will amount to not over 500,000 gallons per day. Taking these figures as a basis for discussion, there appears to be no great reason for alarm when once the two large leaks are closed. Some one, doubtless, was at fault for improper work in sealing the sumps at shafts 1 and 2, but the remedy is comparatively simple and is now being administered by tearing out the old work and doing it over again. So far as the remaining leakage is concerned, 500,000 gallons in 24 hours in a brick-lined tunnel 10 ft. in diameter and 14,000 ft. long under a head of water from 90 to 100 ft. will, we think, appear to engineers to be very moderate. Moreover, this leakage occurs when the tunnel is empty, and, as will appear from the description of the work, published in another column, the conduit will normally have a water pressure on the inside nearly as great as that due to the head of the surface water which is now causing trouble.

The interstate water-rights controversy between Kansas and Colorado has led to a broad-minded stand on the part of the Honorable James Wilson, Secretary of the United States Department of Agriculture. In granting a request that Prof. Elwood Mead, M. Am. Soc. C. E., Chief of Irrigation and Drainage Investigations, be detailed as expert assistant to act with the Assistant Attorney-General of the United States in behalf of the Interior Department, Secretary Wilson has suggested that Professor Mead's services be utilized in settling broad questions of general policy. He considers that it would be unfortunate to have Professor Mead become a witness on one side or the other of any dispute over conditions in a given drainage area.

The Kansas-Colorado case is now before the United States Supreme Court as an original action brought by Kansas asking that Colorado and its citizens be enjoined from appropriating for irrigation purposes the interstate waters of the Arkansas River. Kansas holds that the doctrine of riparian rights should govern in this case and should prevent the diversion of the waters of the river in Colorado. It is urged by Colorado that within its boundaries it has sovereign control of the waters of the river and that subject to its regulations its citizens have the right to all the waters of the river within the State. In the petition for intervention the United States Attorney-General held that neither Kansas nor Colorado was right in its main contentions, but that since the drainage area in question is within the so-called arid region—that is, west of the 99th meridian—the doctrine of prior appropriation, and not that of riparian ownership, should prevail; and this regardless of State boundaries.

Three viewpoints are here presented, one championed by the general government and one each by the two States engaged in the controversy. It may be that the Attorney-General, if he intervenes in such a suit, is compelled to take sides or to make a side of his own, since it is the business of attorneys to take sides. But it may reasonably be expected of the other government departments that instead of prejudging a case, as an attorney must, they will first of all try to get at the facts and the principles involved, and that their single ultimate aim will be to secure the settlement of public questions on the broadest possible basis consistent with equity. In so far as constitutional and common law questions do not govern, this and similar questions should, of course, be settled in accordance with broad views of public policy, not forgetting prospective as well as present interests. The agricultural prosperity of the West, and not of two States merely, is largely involved in this controversy, and therefore the decision should likewise be broad.

It should be remembered that at no very distant day manufacturing interests will demand consideration in some of the arid States that are now thinking of little but irrigated agriculture, and when that time arrives the doctrine of riparian rights will be held in higher esteem. Moreover, the decision in the Kansas-Colorado case can scarcely fail to have some influence upon the great question of the pollution of interstate streams, already in the United States courts. In view of these and other varied interests, it is certainly desirable to have every Washington department that takes a part in water-rights controversies do so in a broad-minded way and carefully avoid taking sides in any controversy unless it is assured that thereby the interests of all concerned will be as well protected as equity administered under fallible human conditions will permit.

The recent failure of wood water-tanks at Upper Sandusky, O., and at Waltham, Mass., through the corrosion of thin flat bands, as noted in our issues of Oct. 27, Nov. 10 and 24, again calls attention to the insecurity of such fastenings. In each case the same size and spacing of bands is reported as having been used at the top and bottom. The moral is: Use good-sized round iron rods instead of thin flat bands for wood tanks, and have their size and spacing proportioned for the strains which they are to bear.

Some interesting tests of pile resistance are given in this issue in an article by Mr. J. J. Welsh. We desire to call attention here, however, to what appears to be an error in Mr. Welsh's statements as to the "settlement" of piles under test loads. His method of making the measurement of settlement is not clear, but it appears quite certain that it was done by taking the elevation of the head of each pile, just before loading and again when under load. If our assumption is correct, then what is set down as the "settlement" of each pile under load must have been largely a settlement due to the compression of the pile fibers under the test load.

Let us apply the well-known formula for elastic compression:

$$(1)\quad a = \frac{P\,l}{A\,E}$$ in which:

a = compression in inches.
P = test load in pounds.
l = length of pile in inches.
A = area of cross-section of pile in sq. ins.
E = modulus of elasticity of pile.

If the pile were a column supported by a rigid foundation at its lower end, the foregoing formula could be applied at once; but, as the piles were supported by skin friction, we must assume that there was little or no compression of the fibers at the toe of the pile. If the pile were uniform in diameter and the friction was uniform per equal foot of surface in contact with the mud, the compression would increase uniformly from zero at the toe to the maximum compression at the top or butt; hence, the equivalent length of the pile would be ½ l. The formula to apply is, therefore,

$$(2)\quad a = \frac{P\,\frac{1}{2}\,l}{A\,E}$$

$l = 70 \times 12 = 840$ ins.; $\frac{1}{2}\,l = 420$.
$A = 80$ sq. ins., or less.
$E = 1{,}700{,}000$ for Douglas fir.
$P = 44{,}800$ lbs., first day.
$P = 80{,}000$ lbs., second day.

$$(3)\quad a = \frac{44{,}800 \times 420}{80 \times 1{,}700{,}000} = 0.14 \text{ ins., or nearly } 1\text{-}7 \text{ in.}$$

$$(4)\quad a = \frac{80{,}600 \times 420}{80 \times 1{,}700{,}000} = 0.25 \text{ in., or } \tfrac{1}{4}\text{-in.}$$

Hence, the total fiber compression of each pile under the first day's loading should be about 1-7 in.; and, referring to the table given by Mr. Welsh, we find that it was actually 1-8 in. for each and every pile.

The total fiber compression of each pile under the second day's loading should be ¼-in.; and, by the table it was 1-8 + 1-8 = ¼ in. for pile No. 1. For the other three piles the "settlement" was greater, showing that there was probably some true settlement in addition to the fiber compression.

If there were any doubt as to the correctness of our assumption that fiber compression, and not true settlement, was measured under the load of 44,800 lbs. per pile, that doubt would be lessened by the fact that the so-called settlement was exactly ⅛-in. for each and every pile. If the piles had actually settled under their load, it is not at all likely that they would have settled so uniformly, for these piles were driven in very soft mud.

Mr. Welsh fails to state how long a time intervened between the striking of the last blow and the placing of the first test load; but doubtless the time was long enough to permit the mud, stirred up by the driving, to settle around the pile and greatly increase its bearing power. This inference is supported by a study of the table wherein it appears that pile No. 1, driven with a drop hammer penetrated 6 ins. under a blow having an energy of 36,960 ft.-lbs., while pile No. 3, driven with a steam hammer penetrated precisely the same number of inches, 6 ins., under a blow having an energy of only 17,500 ft.-lbs. It is true, of course, that the dead weight of the steam hammer frame which rests continuously on the pile assists in pushing it down; but this assistance must have been merely nominal at the last blow, for the hammer frame weighs only 9,850 — 5,000

= 4,850 lbs., which is only 10% of the test load that produced no true settlement at the end of the first day's test. Every blow of a hammer stirs up the mud around the pile, and thus reduces its grip upon the pile. The more rapid the delivery of the blows the less the frictional resistance, and consequently the greater the penetration under each blow. Comparing the 36,960 ft.-lbs. of energy in the last blow on pile No. 1, with the 17,500 ft.-lbs. on pile No. 3, we are led to conclude that the rapid blows of a steam hammer reduce the skin friction of the silt on the pile by at least 50%.

However, no safe generalization can be drawn from one set of tests, and our purpose is merely to call attention to some of the reasons for the different results obtained by the use of the Engineering News formula. For the piles driven with the drop hammer the formula is seen to give a factor of safety of 7.6 for pile No. 1, and a factor of safety of about 4.5 for pile No. 2, since neither pile showed any true settlements until a load of about 80,000 lbs. was applied. The Engineering News formula, it should be remembered, contains a factor of safety of 6.

For piles Nos. 3 and 4, the Engineering News formula gives much higher factors of safety, in one case 14 and in the other case 9½, where a true settlement of about ¼-in. is admissible, but we have no means of knowing what the actual factor of safety was at the moment that settlement began.

A new field for young engineers and for sanitary chemists and biologists has been slowly opening of late and now promises well for the future. We refer to the positions of executive health officer and sanitary inspector under local boards of health. New Jersey has led the other states of the Union in passing a law providing that all health officers and sanitary inspectors appointed after Jan. 1, 1905, shall hold certificates granted by its State Board of Health after passing examinations prescribed by the board. Notice of an examination on Dec. 7 was given in our Supplement last week. Next June another examination will be held. We desire to bring these examinations to the attention of the teaching staffs and the students of such of our technical schools as give instruction in municipal sanitation. It would be well if professors and students alike would consider the possibilities of courses in municipal sanitation, and it is to be hoped that some good men from our technical schools will present themselves for examination at Trenton, N. J., on Dec. 7.

VENTILATION OF THE NEW YORK SUBWAY.

The New York Rapid Transit railway has now been in operation for a month, and has carried in that time over six million passengers. It has been run more smoothly and with less interruptions and delay than commonly falls to the lot of a new railway, particularly one involving so many radical departures from existing practice, and operated under such unusual conditions. It has been performing a public service which the vast majority of its hundreds of thousands of its passengers fully appreciate. It is a curious fact, nevertheless, that since the echoes of the congratulatory speeches over the opening of the subway died away, the public attention has been wholly taken up with the two matters of impure air in the subway and advertising placards on the walls of its stations. So far as the newspapers reflect public sentiment, it would appear as if Father Knickerbocker, like a spoiled child with a new toy, had forgotten entirely, for the moment, the value of his prize in anxiety over some pin-holes in its varnish.

When the subway was first opened to traffic, there was considerable odor perceptible to one entering it, and this was natural enough, considering the amount of paint, cement, and other materials which had been recently placed, to say nothing of the odor from machinery and oils used in its lubrication. Something was said in the newspapers concerning the ventilation of the subway, and then a physician, Dr. August Siebert, by name, undertook to analyze samples of the subway air and published results which showed a large deficiency in oxygen. We quote as follows the substance of Dr. Siebert's published statement as it appeared in the daily papers:

> While taking my first ride in the subway, two days after the opening, I noticed an increase in my respiration from 20 to 28 per minute, after resting quietly on my seat in a half-filled car for ten minutes. This proved air-hunger. To test the accuracy of this assumption I bought an oxygen apparatus of one of the best-known firms in town. The head of this concern was kind enough to put this Lindemann apparatus himself in proper condition and to instruct me as to its use.
>
> Air was taken into a glass cylinder filled with 300 cubic centimeters of sterilized and filtered water by allowing the water to flow into a bottle at the lower end, the air to come in at the upper end. Then both ends of the cylinder were closed by airtight stopcocks.
>
> These samples of subway air were taken at the end of the platforms of different subway stations between 103d St. and City Hall.
>
> At home 100 cubic centimeters of the subway air was introduced into Lindemann's apparatus for estimating oxygen and therein deprived of its oxygen by contact with phosphorus, so that the exact amount taken from this air could be read off on the graded tube.
>
> In all, twelve tests were made, each verified by a second controlling test, and for comparison the oxygen of the surface air was also determined. The tests are as follows, in the order made:

Station.	Date.	Time.	P. c. Oxygen.
103d St.	Oct. 30	10.30 a. m.	12.0
103d St.	Oct. 31	11.30 a. m.	12.0
103d St.	Nov. 1	4.00 p. m.	11.9
14th St.	Oct. 30	5.00 p. m.	12.1
14th St.	Oct. 31	10.00 a. m.	12.0
14th St.	Oct. 31	4.00 p. m.	12.1
33d St.	Nov. 1	10.00 a. m.	12.5
33d St.	Nov. 1	4.00 p. m.	12.3
33d St.	Nov. 2	9.00 p. m.	12.0
City Hall	Nov. 3	10.00 a. m.	16.5
City Hall	Nov. 3	4.00 p. m.	17.0
City Hall	Nov. 3	9.00 p. m.	16.5

> The surface air during these five days was dry and cool, and the oxygen therein varied between 19.6% and 20.1%. Allowing for small inaccuracies, this brings our surface air very near to the standard of 20.8% of oxygen.

Naturally, the ordinary newspaper reader having his attention first attracted by such "scare headlines," as, "Subway Air Dangerous, Says Doctor After Test," accepted Dr. Siebert's pseudo-science as if it were the real thing. The newspapers also appeared to generally accept it, and up to the present time the published protests of several chemists of reputation have not seemed sufficient to shake the public confidence in Dr. Siebert's so-called analyses.

It is, of course, well known to chemists and sanitary engineers that the test in universal use for determining the quality of air in connection with ventilation problems is the carbonic acid test. The very fact that Dr. Siebert used some other method than this for testing the subway air cast doubt on his results at the outset.

Some chemist pointed out this fact in the daily press, and Dr. Siebert's reply to this criticism was highly amusing. We quote as follows:

> To guess at the actual amount of oxygen by determining the amount of carbon dioxide is insufficient. Why not try the Lindemann apparatus to ascertain the actual amount of oxygen? Why use this roundabout way? The number of saddles hanging in a stable do not actually give the number of horses. Why inferences, when actual tests for oxygen are possible?

It will be noted that by Dr. Siebert's own statement he was himself so unfamiliar with the chemical apparatus which he used that he had to be instructed as to the method of operating it. Yet he had the presumption to apply a method not in common use for the analysis of air and spread its results before the public. By inference, also, he was so unfamiliar with the usual standard method of testing air for impurities that he never thought of applying it to the subway air, but undertook to determine the oxygen percentage in the air directly, in no whit disconcerted by his own lack of familiarity with chemical apparatus and methods.

If any further proof were needed as to his limited experience in chemical work, it is furnished by the paragraph quoted above. If it is "guessing" at the amount of oxygen in a closed room to determine the amount of carbon dioxide contained, then nearly all chemical analysis is guessing, for it very seldom happens that elements can be isolated directly, it being nearly always necessary to weigh or measure them when in combination.

Of course, it is well known that carbonic acid gas itself is not particularly injurious, in such quantities as occur in badly ventilated rooms. It is because it is a convenient index of the extent to which the air is polluted that the carbonic acid test has long been universally accepted by sanitarians.

In this connection it is of some interest to note what Dr. Siebert's tests, had they been correct, would have signified as to the percentage of carbonic acid gas present in the subway air. Ordinary air contains about 20.8% of oxygen, about 79.2% of nitrogen, and from 3 to 4 parts in 1,000, or 0.03 to 0.04% of carbonic acid gas. In breathing, the oxygen taken into the lungs combines with carbon and is exhaled as carbonic acid gas. If the subway air is deficient in oxygen, as Dr. Siebert claims, it must be because the oxygen has been converted into carbonic acid gas, and if in any part of the subway the percentage of oxygen in the air has been reduced to 12% instead of the normal of about 20.8, then at that same place there should be found about 7½% of carbonic acid in the air. But 7½% of carbonic acid is equivalent to 75 parts per 1,000 or 750 parts per 10,000, and this exceeds by far the highest records of carbonic acid in air taken in the most densely crowded assemblies. Billings, in his well-known treatise on ventilation, sets 30 to 40 and "even 100 parts" per 10,000 as the maximum carbonic acid contained in such confined spaces as the pit of a crowded theater.

It seems perfectly certain, therefore, that Dr. Siebert simply made a huge blunder through unfamiliarity with chemical methods and manipulation, and reported a condition of the air in the subway probably ten times or a hundred times worse than its actual condition.

That such absurb statements, bearing evidence of falsity on their face to any one with even an elementary knowledge of chemistry, should have been accepted and given publicity by the metropolitan daily papers, seems indeed strange; but stranger still is the fact that a journal devoted to heating and ventilation was so impressed by Dr. Siebert's published figures that it printed the following sage comment:

> The subway is by no means incapable of adequate ventilation. The stations are already fairly ventilated, and certain portions of the tunnel are practically open to the air. If it is true that the proportion of oxygen has already been reduced below 11%, as is stated by some who have made an analysis of it, it has been brought much below the line of salubrity.

"Much below the line of salubrity" shows such extreme caution, that one wishes the editor had explained just where he thinks the "line of salubrity" should be drawn.

And now, having duly recorded the nonsense which has been printed concerning the air of the subway, our readers will doubtless be interested to learn what are the real facts. First, actual tests by Prof. C. F. Chandler of Columbia University have shown, as might be expected, that any pollution of the subway air by respiration is much too small in amount to be determined by an analysis of its oxygen percentage.

In the second place the subway air, judging by the senses, is less polluted by far than the air which New Yorkers are accustomed to breathe in many public places. It will compare very favorably, for example, with the air in elevated railway cars during the rush hours. It is purity exemplified compared with the noxious atmosphere which some hundreds of thousands have to breathe daily in the cabins of North and East River ferry-boats.

It is probable that during the next few months, while the winds of winter are prevailing and the air in the subway is warmer and lighter than the air in the streets above, there will be little complaint and little ground for complaint of the subway air. When spring comes and the temperature overhead exceeds that of the subway, the natural ventilation currents will be greatly reduced, and unless the public has become accustomed to the subway by that time so as to accept bad air there as it does elsewhere without a murmur, we may hear then renewed public demand for subway ventilation.

In our opinion, provision for ventilation should be made in any such subway, and we understand it has been made in the New York subway and all that remains to be done is to install fans and electric motors in the ventilation passages.

A little computation will show that the power required to furnish the standard amount of 3,000 cu. ft. of fresh air per hour for each person in the subway, would require a very moderate installa-

tion of electric motors and fans, involving nothing of all excessive either in the cost of installation or of operation. Such a positive ventilation, moreover, would change the entire subway air at least once every 24 hours, and would probably be sufficient to prevent stagnation, mustiness and general odors, which have been the chief sources of complaint in the subway air thus far.

If such liberal ventilation were afforded, moreover, the air from the ventilating shafts could be discharged at the street level or, at most, at the level of the house-tops without producing any nuisance or cause of complaint.

We are well aware that when underground electric railways were originally projected, it was thought that the elimination of the steam locomotive and the gases of combustion and vapor would make artificial ventilation of such tunnels unnecessary. Experience in the London deep tunnels, however, showed that the stagnant air in these tunnels became rapidly fouled until the odors in these tunnels were worse by far than on the old London underground lines operated by steam locomotives.

It may safely be set down, therefore, that at the present time artificial ventilation should be provided on all underground railways, notwithstanding the use of electricity for traction and lighting.

LETTERS TO THE EDITOR.

A Weak Horse-Power and a Magic Pulley!

Sir: I enclose a page from the advertising section of "Everybody's" for December. It would appear that advertising matter dealing in any way with engineering subjects should be prepared by those knowing at least the definition of the most common words in the engineering vocabulary.

The talented author of the enclosed "ad." seems to believe also that the tractive force will be equal to the

Touring Car
What 16 Horse Power Means
528,000 Lbs.
The Mechanical Definition of 1 H. P.
means the power necessary to lift 33,000 pounds one foot per hour. Sixteen actual horse-power would therefore lift 528,000 pounds one foot an hour.

full power of his 16 I. HP. (?) engine. He would probably change his mind if he were stuck on a red clay hill in a rain storm. At such times the two-horse power of the farmer's "plugs" is likely to be worth more than the hyphen-power of the gas engine, even if applied through such an ingenious (?) mechanical device as shown in the illustration.

Respectfully,
Harland M. Kerschner.

Elmira, N. Y., Nov. 22, 1904.

(We reproduce the ad. herewith (omitting the name of the machine). The artist who drew the picture was evidently clear that the locomotive would require very many strands of rope to sustain its weight, while a single rope would hold all the automobile could pull; but his ideas as to how ropes and pulleys could be arranged to enable the little motor-car to lift the big weight were probably so hazy that he put in the simplest possible "funicular machine" and let it go at that.

As for the "ad. writer," he has now an opportunity to explain that his machine is really capable of lifting a weight of 3,168,000 lbs. at the rate of 1 ft. per hour, instead of the modest little weight previously claimed!—Ed.)

The Action of Water on Lead Service Pipe in Teplitz-Schonau, Bohemia, and in Massachusetts.

Sir: I have read with considerable interest the article in your issue of Nov. 10 in regard to lead in the water of Teplitz-Schönau, in Bohemia, and the method proposed for preventing this water from acting upon lead pipe.

I presume that in the "sulphurized" lead pipe, lead sulphate is eventually formed, which is a very insoluble body, and this sulphate serves as a protective coating against the action of the waters. In this laboratory we found, a few years ago, when a certain waste pipe had to be replaced, that the trap of this pipe had become, owing to the constant presence of sulphuric acid in it, coated with lead sulphate and was well protected and whole, while below this trap the pipe was badly eaten.

The proposed hardening of the water by causing carbonate and sulphate of lime to be taken into solution is a natural process and one resorted to at many places in England and on the Continent. The proposed addition of carbonic acid to the water before hardening is simply that larger amounts of these bodies may be taken into solution to become subsequently deposited upon the pipes. That is to say, the water will be made so hard that even the presence of considerable free carbonic acid will not cause lead to be taken into solution.

In 1901, the Local Government Board of Great Britain issued a special report on "Lead Poisoning and Water Supplies." In this report illustrations are given of the methods resorted to at various English municipalities to prevent the action of their water supplies upon lead. To quote from the report, we find the following instances:

Wakefield Corporation Water-Works treat their water, which previous to treatment is strongly acid and possessed of great plumbo solvent ability, with a solution of sodium carbonate; 500 gallons* of a strong solution of soda (about 1 lb. per gallon) are mixed with 1,000,000 gallons of acid water.

At Barnsley the water is filtered. The filtering material is made up of layers of sand, grit-stone and limestone.

At Sheffield the acid water from the high-level gathering ground is treated with lime (about 2 grains to the gallon) in an apparatus which works automatically, and thoroughly mixes the water and lime together.

At Rochdale there are two reservoirs. The upper reservoir is acid, and the water is treated in the lower reservoir with lime by means of boats, the lime being thrown by means of scoops on the surface of the water.

At Bradford the water from Thornton Moor Reservoir, which is acid and dissolves lead, is treated with pure carbonate of lime (about 3 grains to the gallon) and is then filtered through sand and gravel.

At Morley many of the inlet streams to Withen's Clough Reservoir have small filter beds of stone and sand covered with limestone and chalk.

In the articles, to which you refer, published in the reports of the Massachusetts State Board of Health attention is drawn to the fact that in pure soft waters both oxygen and carbonic acid attack lead. When, however, water is soft and contains considerable oxygen and not much carbonic acid the pipe becomes coated in the interior, while when the reverse is the case (that is, comparatively small amounts of oxygen and much carbonic acid is present in a soft water) the pipe is kept comparatively clean and continually attacked by the acid. In the article in the report for 1900, the following statement was made:

Many laboratory experiments during the past three years have shown that while pure soft water, especially when containing some dissolved oxygen, attacks lead, and while the presence of coloring matter, free ammonia, nitrites and nitrates in soft water also causes considerable solvent action upon lead in laboratory experiments, yet taking into consideration the results of our *entire* investigation, we find that in actual practice, with the conditions prevailing in the service pipes of a distribution system, a potable water in *Massachusetts* to have any dangerous lead-dissolving action must contain considerable free carbonic acid.

That is, we concluded from the results of our entire work, not only laboratory experiments but examinations of water from the different towns and cities, of service pipes, and investigations in regard to cases of lead poisoning, that while many bodies (oxygen, nitrites, nitrates, etc.), attacked lead, yet in the conditions prevailing in this State, carbonic acid in a soft water was the main factor that caused lead to be taken into solution. Professor Gintl's conclusion is that lead poisoning by water "is directly caused by the dissolved air." As a matter of fact, the surface water supplies of Massachusetts are always practically saturated with air, but these waters do not attack the service pipes. On the other hand, our ground waters, containing less dissolved oxygen than the surface waters but much more carbonic acid, do attack lead service pipes and it is in municipalities with such supplies that practically all our cases of lead poisoning caused by drinking water have occurred.

Yours very truly, H. W. Clark.
Chemist, Massachusetts State Board of Health.

Boston, Mass., Nov. 22, 1904.

*These are doubtless Imperial gallons; add one-fifth for the equivalent in U. S. gallons.—Ed.)

Why Do Some Concrete Walls Crack While Others Do Not?

Sir: Your correspondent, "Ashlar," asks in your issue of Nov. 17: "How can cracks be prevented in the ashlar facing of a concrete wall," to which you reply.

The reply induces me to ask a question. Why do some concrete walls of great length crack and some do not? I have seen lock walls built in the Coosa River of concrete on a magnificent rock foundation, with cracks in them large enough to insert one's hand. These cracks appeared at distances apart as near as I can now remember of about sixty to seventy feet. They looked as though the concrete walls had contracted in length some inches, breaking the walls into large sections. These walls, as I say, were built on rock foundations and the cracks were not the result of its unequal settlement.

A few years ago I had occasion to visit the Canadian dock in Liverpool. As near as I can remember, the side walls were somewhat over 1,000 ft. in length, measured from out to out. I was credibly informed that each wall was a monolith, built entirely of concrete, and resting partly on a rock and partly on a hard-pan foundation. There was not the sign of a crack in either side wall that could be detected by ordinary eyesight. Each wall was like a single stone. I was informed that no cracks had ever appeared in these walls (certainly there were no indications that any had ever been filled up)—and that no special devices had been used in laying the walls to prevent shrinkage. Yet the walls appeared without a blemish.

The walls of this dock as well as the walls of the lock before referred to were built of Portland cement concrete, but as to the relative proportions of the ingredients, I cannot say. Here we have two concrete walls, built of much the same material, both on good foundations, but the one on the best foundations was badly cracked while the other, though nearly three times as long as the first, was not cracked at all.

Can you offer any explanation?

Peter C. Hains, M. Am. Soc. C. E.

Washington, D. C., Nov. 18, 1904.

(This interesting question of cracking of concrete walls has never received the investigation that it merits by virtue of its importance. There are apparently two theories as to the force that causes cracking: (1) Shrinkage of the cement in setting; and (2) contraction of the wall due to temperature changes. If we seek to find any tests made to determine the shrinkage of concrete upon hardening we shall look in vain. A few tests have been made upon the shrinkage of neat cements and upon mortars, but none, so far as we know, upon concrete. The shrinkage of a 1:2 or 3 mortar allowed to harden in the air in small cubes, is about 1 part in 2,000, which is about half as great a shrinkage as for neat cement. What is the shrinkage of a 1:3:6 concrete made of broken stone, or gravel, or a mixture of both, made in large masses and exposed to the air? We do not know. Will a concrete made with an excess of water shrink more or less, upon hardening, than one made "dry"? We can not say. All we know is that small blocks of cement mortar allowed to set under water do not shrink, but actually expand slightly upon hardening.

If a 1:3:6 concrete wall, 1,000 ft. long, were to shrink as much as a 1:3 mortar we should have a shortening of 6 ins., but this shorting would be certain to open up cracks, and these cracks would remain permanently open if there were no change in temperature. A rise of temperature of nearly 100° F. would be required to close the cracks;

for, with a coefficient of heat expansion of .000006, a rise of 100° F. in a wall 1,000 ft. long would produce an expansion of 0.6 ft. We know, however, that vertical cracks in concrete retaining walls usually close up entirely in hot weather, even where the rise of temperature is not much above the temperature at which the concrete hardened. Evidently, then, concrete walls are either not cracked by the shrinkage of the cement, or if they are, the shrinkage is far less than ½ ft. in 1,000 ft.

In the Boontown Dam, N. J., experiments are now in progress to determine the temperature of the rubble concrete masonry at various distances from the surface of the masonry. It has been found that the temperature of the concrete rises steadily after deposition, reaching a certain maximum which is independent of the temperature of the outside air. This rise of temperature is, of course, due to the chemical action that occurs when cement hardens. It is apparent, we think, that a small cube of mortar will radiate this heat of chemical combination at such a rate as to experience no internal rise of temperature equal to that in a large mass of masonry. Hence we can not safely draw conclusions as to shrinkage of concrete based upon tests made upon small cubes of mortar.

It is probable that even "dry" concrete in mass shrinks far less upon hardening in the air than do small cubes of mortar. Furthermore, it is probable that concrete deposited very wet shrinks less than concrete deposited dry—possibly it may expand, its cement behaving in the presence of an excess of water just as mortar behaves when hardened under water.

Our correspondent cites a concrete wall 1,000 ft. long, presumably monolithic, and yet a wall that has never developed a crack. We do not know how it was built. If the concrete was deposited under water, it would not crack for it would expand slightly upon hardening. But even assuming that this concrete was deposited in the open air, if shrinkage cracks did not develop upon hardening, temperature cracks have had no chance to develop since the hardening, because the wall has been submerged with the exception of a few feet near its top. A wall almost entirely submerged in sea water is evidently not subjected to changes of temperature that will crack it so readily as a wall entirely exposed to the sun. Should such a wall be exposed after months of submersion, still it might not crack due to an acquired tensile strength that might resist the stress developed by temperature changes.

We are inclined to believe that cracks in concrete walls are due to temperature stresses, rather than to stresses developed by the shrinkage of cement upon hardening; if for no other reason than that cracks usually occur vertically and at quite regular intervals, and that they open and close with variations in the temperature.—Ed.)

Notes and Queries.

The dynamite explosion at Mt. Vernon, N. Y., recorded in our issue of Nov 10, proves to have been a much less serious affair than was at first stated by the daily press, from which information our report was prepared. The facts as reported to us from an authoritative source were as follows: A 50-lb case of dynamite was prepared for use by the man in charge of the explosives, and was placed in a small tool box under a highway bridge while he ate his lunch. Later he discovered one end of the box to be on fire, probably from some one dropping a match or cigar from the bridge overhead upon dry leaves near by. He ran for water to put out the fire, but the dynamite in the box exploded before the water was applied. The explosion tore the wooden floor from the bridge, cracked ceilings in houses near by and broke a considerable number of windows in the vicinity, causing some slight injuries to several persons by flying glass.

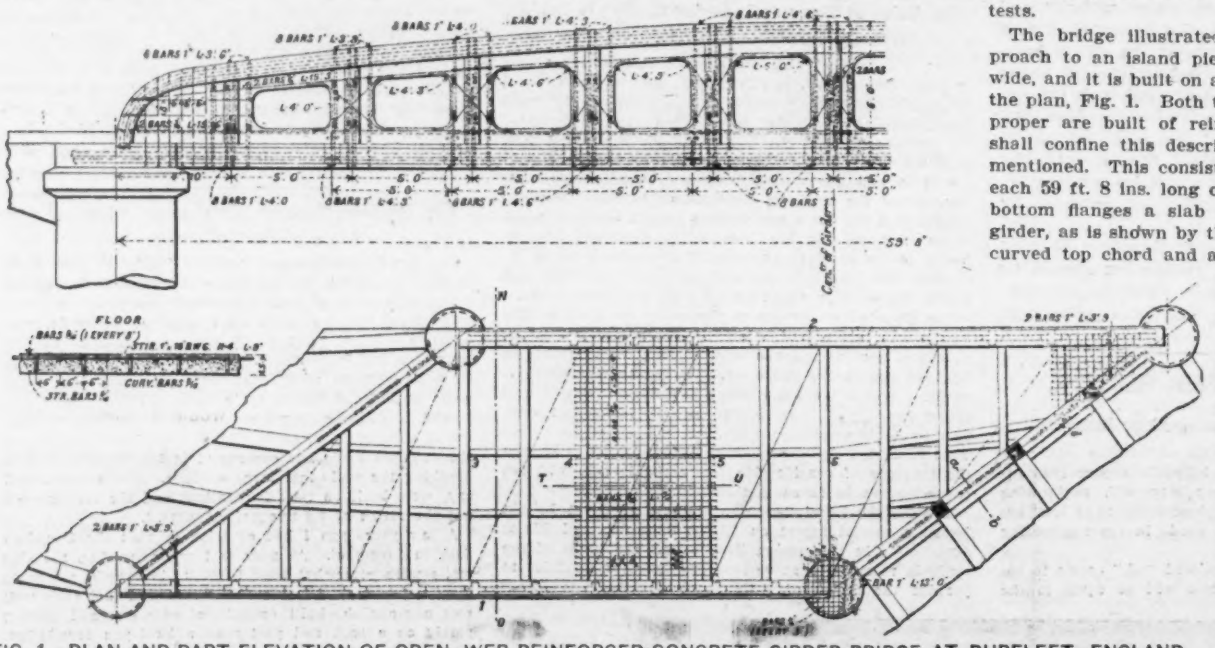


FIG. 1. PLAN AND PART ELEVATION OF OPEN WEB REINFORCED CONCRETE GIRDER BRIDGE AT PURFLEET, ENGLAND.

A REINFORCED CONCRETE GIRDER BRIDGE WITH OPEN WEBS.

Reinforced concrete girders of 50 to 60-ft. span have become so common in bridge and building construction that they have almost ceased to attract attention from engineers familiar with this type of construction. The girders heretofore built, however, have been almost invariably of solid beam form, carrying the load on the top flange, and for this reason the reinforced concrete girders illustrated here and recently used in a small bridge span at Purfleet, England, possess decided novelty because of their open-web construction. Our English contemporaries, to whom we are indebted for accounts of the structure, are silent as to the reason for the open-web design, and it is fairly open to doubt, we think, whether it possesses any very good reason for existence. However this may be, it is of interest to note the successful construction of these girders and their satisfactory behavior under rather severe loading tests.

The bridge illustrated forms part of the approach to an island pier 250 ft. long and 34 ft. wide, and it is built on a sharp skew as shown by the plan, Fig. 1. Both the approach and the pier proper are built of reinforced concrete, but we shall confine this description to the girder span mentioned. This consists of two parallel girders, each 59 ft. 8 ins. long over all, carrying on their bottom flanges a slab and girder floor. Each girder, as is shown by the drawings, consists of a curved top chord and a horizontal bottom chord connected at intervals by vertical posts of cruciform cross-section. A half elevation of one of the girders is given by Fig. 1, and Fig. 2 shows an enlarged detail of a part of one of the girders. The following further description of the construction is taken from London "Engineering" of Oct. 28, 1904:

> The upper chord is 18 ins. wide by 9 ins. deep, and is reinforced by eight 1⅛-in. round bars (Fig. 2). Cross-bars are inserted transversely at frequent intervals, and serve to check any tendency of the concrete to bulge laterally when the load comes on the bridge. The lower or tension chord is also stiffened by eight 1¾-in. round bars, which, as indicated, are connected together by stirrups of No. 12 S. W. G. iron 2 ins. wide. These stirrups are placed at intervals of every 10 ins. and add much to the solidity of the compound chord. The verticals are cross-shaped in plan, each arm of the cross being 7 ins. long by 3 ins. thick. Vertical bars, having hooked ends which lap over the main bars in the upper and lower chords, are used to strengthen these verticals. To avoid delicate adjustments these vertical stiffening-bars are made in two parts, which, after putting in place, are securely bound together by iron wire, as indicated. Along the top and bottom edges of each panel-opening strength is afforded by two ¾-in. bars extending from end to end of the girder. Iron reinforcing-bars are carried round each corner of the panel-opening (Fig. 2), and prevent any tendency of the concrete to crack or open at these corners under the shearing stresses. The decking is also of reinforced concrete, and together with the main trusses forms one monolithic structure. The cross girders are 15 ins. deep and 8 ins. wide. The upper reinforcing bars have hooked ends reaching to the central sections of the main trusses. The lower ones are carried through the chord

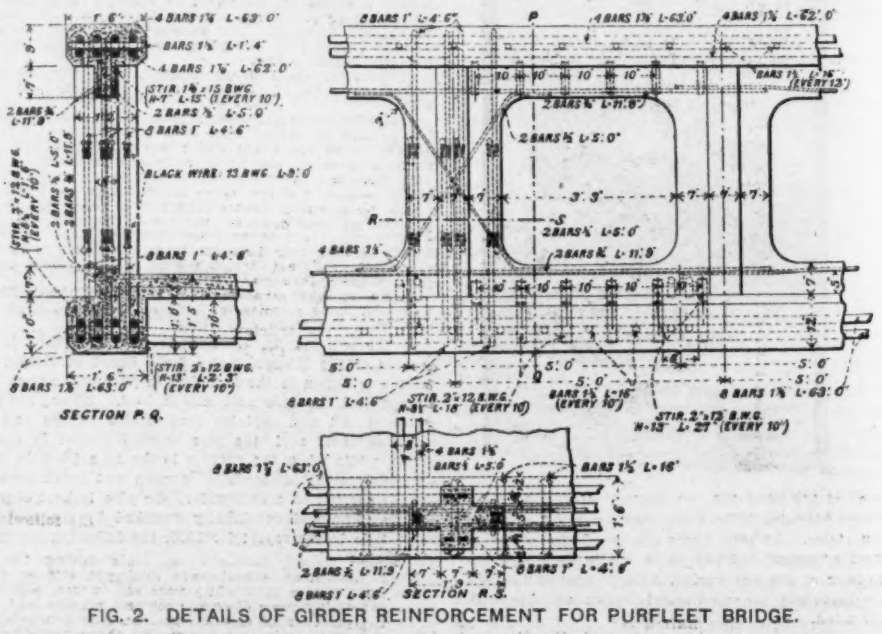


FIG. 2. DETAILS OF GIRDER REINFORCEMENT FOR PURFLEET BRIDGE.

between the two rows of 1¾-in. bars, replacing the transverse stiffening-bars used at intermediate points. In all cases sheet-iron stirrups support these strengthening bars at frequent intervals, the idea being to bind the concrete together transversely as well as in the direction of the main stresses.

The decking between the cross girders is 3 ins. thick, and is, of course, thoroughly reinforced by embedded steel bars. The bridge carries a single line of standard gage railway, and immediately under the rails the decking is stiffened by longitudinals, as indicated in Fig. 1.

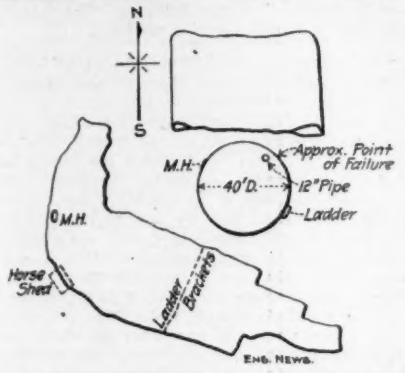


Fig. 1. Sketch Plan of Wrecked Stand-Pipe, Sanford, Me.

The total weight of the bridge is given by the same authority as 90 tons, of which 15 tons are credited to the steel reinforcement. The concrete was a one Portland cement and four ¼-in. broken stone mixture. The bridge was tested by two cars with bogie trucks and 24-ft. wheelbase, loaded to a weight of 30 tons each. Under this load the maximum deflection was 0.197-in.

The bridge was built by the agency of the Hennebique Co. in England, and Mr. P. W. Meik was consulting engineer.

A STAND PIPE FAILURE AT SANFORD, ME.

By Charles W. Sherman,* M. Am. Soc. C. E.

The 40 × 80-ft. steel stand-pipe of the Sanford Light & Water Co., Sanford, Me., failed early on the morning of Thursday, Nov. 17, 1904, and the 750,000 gallons of water which it contained rushed down the hillside without causing damage of any moment.

The water-works were built a number of years ago. The water was originally pumped to a small wooden tank in the village. About seven years ago a steel stand-pipe, 40 ft. in diameter by 60 ft. high, was built on the hill north of the town. It was constructed by the Portland Co., of Portland, Me., and it is understood that no engineer was employed. At that time the fire pressure was obtained by pumping into the mains with mill fire-pumps, but with the rapidly increased pollution of the Mousam River it became unsafe to pump the river water into the mains, and it therefore became necessary to have a stand-pipe at a higher elevation. Instead of constructing a new tank, the old one was taken down, the rivets being cut out, and moved to a higher hill on the opposite side of the valley, and reerected; and at the same time 20 ft. additional was added at the top. This removal and rebuilding was also done by the Portland Co., and was completed about seven weeks ago. Since that time the stand-pipe has been in use, but not until the night of its failure was it entirely filled. The height of the water ranged between 8 ft. and 2½ ft. below the top until 10.30 p. m., Nov. 16, when the tank was filled to the top. The failure occurred at about 3 a. m., Nov. 17.

After its rebuilding the stand-pipe leaked somewhat and had been calked, but without entirely stopping the leak.

It is understood that the Sanford Light & Water Co. has had an expert examining the wreck, and until it receives his report it does not care to give out any information. An examination of the ruins, however, discloses substantially all the interesting facts, with the exception of the character and composition of the steel. This examination was made by the writer on Nov. 22, five days after the failure.

The foundation was of rubble masonry in cement, extending to solid rock. There were no anchor rods. The foundation and bottom of the tank are still substantially intact.

The bottom plates are 7-16-in. thick, and the bottom angle, which was outside the shell of the tank, is 5 × 5 × 9-16-ins., double riveted to both bottom and shell. The courses of the shell were each 5 ft. high, each course being within that below, thus presumably making the diameter somewhat less at the top of the tank than at the bottom. The vertical joints were lapped and double riveted in the lowest six courses and single riveted in the remainder. All horizontal joints were single riveted. The rivet spacing in both vertical and horizontal joints was 2½ ins. in all single riveted joints; and in the double riveted joints the rivets were 3½ ins. apart in the rows and staggered, the rows being 1¾ ins. apart.

The thicknesses of the plates in the several courses were found to be as follows:

Course.	Thickness, ins.	Course.	Thickness, ins.
1	7/16	9	9/16
2	7/16	10	9/16
3	9/16	11	5/16
4	9/16	12	5/16
5	1/2	13	5/16
6	7/16	14	5/16
7	3/8	15	5/16
8	3/8	top	1/4

The top of the tank was stiffened by a 3 × 3 × ¼-in. angle on the outside. There was an outside iron ladder, and a manhole in the bottom course, the approximate position of which are shown in the sketch plan, Fig. 1. The admission and discharge pipe was 12 ins. in diameter, and entered at the bottom of the tank approximately at the location shown in the plan. Figs. 2 and 3 are general views of the wreck.

The lower half (exactly eight courses) of the stand-pipe was split vertically, unrolled and projected about 50 ft. to the southwest of the foundation, falling with the outside uppermost. The upper half simply toppled over and fell to the north or northeast, just clearing the foundation, and of course collapsed as it struck the ground.

It seems evident that the point of the initial rupture was in the northeast side of the tank, and in one of the two lowest courses, and that from this point the steel was torn vertically to a height of 40 ft. The tension in the lowest course with a full stand-pipe would be about 13,000 lbs. per sq. in. on the full section, and about 16,000 lbs. per sq. in. on the net section, deducting rivet holes. Many of the fractured edges were rusted to such an extent that it was difficult to tell much about the metal, but enough clean fractures were found to indicate that the steel was hard and brittle, showing a crystaline structure. Apparently no rivets were sheared; many plates were torn through the rivet holes and many were ruptured at considerable distances from the edges. *A number of rivet holes were found where there were one or more cracks radiating from the* hole. One such is shown in Fig. 4. It is worthy of note that the rivet holes examined showed no evidences of drifting. Some of the fractures in the centers of plates are evidently due to blows received when the plates struck rocks or trees, and others have resulted from bending as the immense section fell to the ground; several of these latter appear to have resulted from bending through angles much less than 180 degrees and with radii greater than 12 ins. Still other fractures, both vertical and horizontal, at considerable distances from the edges of plates, show no indication of their origin. One fracture was noted in which the break was nearly at right angles to the face of the plate for about half its thickness, then parallel to the face for about 5/8-in., and then at right angles for the remainder of the thickness.

It seems probable that the rupture originally started in a crack radiating from a rivet hole;

Fig. 2. Looking Nearly Southwest from Bottom of Tank.

Fig. 3. Looking West; Showing Buckled Portion of Lower Section.

FIGS. 2 AND 3. VIEWS OF WRECKED STAND-PIPE AT SANFORD, ME.

Fig. 4. View Showing Cracks Extending from Rivet Holes of Wrecked Stand-Pipe.

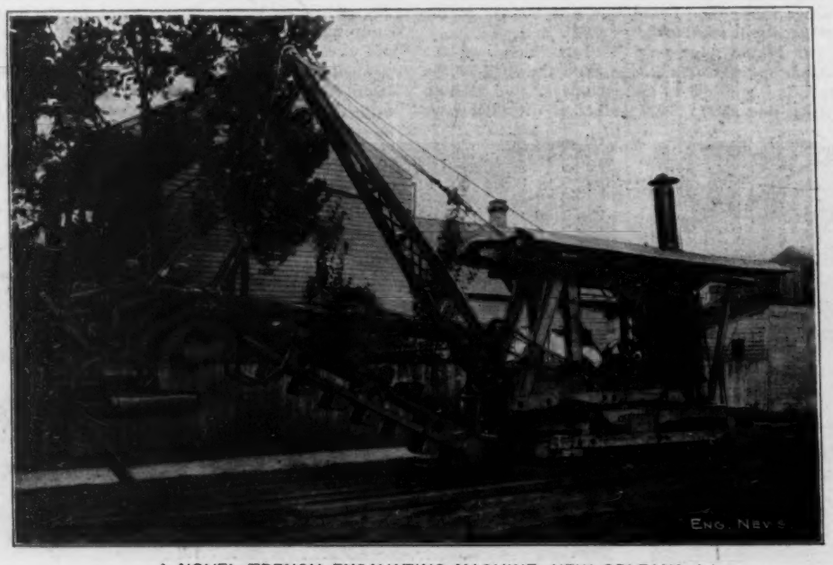
*Assistant Engineer with Leonard Metcalf, M. Am. Soc. C. E., Consulting Engineer, 14 Beacon St., Boston, Mass.

and that these radiating cracks may have been caused in the brittle steel by the excessive hammering due to cutting out the rivets, reerecting the tank, and repeated calking of leaky joints. It is not evident, however, why the failure did not take place immediately upon the application of the full pressure.

A NOVEL TRENCH EXCAVATING MACHINE.

A trench excavating machine that has been successfully used on sewer work at New Orleans, La., is shown in the accompanying photograph. The method of operating this machine differs from all others of its kind in that no attempt is made to cut the full width of the trench at one stroke. Instead, three slices are cut by the scoop or digger in excavating a trench 6½ ft. wide. To do this, the upper or machine platform is movable transversely on the lower or truck platform which rides on a track.

As seen in the photograph, the machine consists essentially of an endless bucket elevator which delivers the earth upon a transverse belt conveyor which in turn delivers the earth into cars or wagons or piles it alongside the trench. At the lower end of the bucket elevator is a scoop provided with teeth, a bail and a cable leading to the hoist. As this scoop is pulled up along the face of the trench it cut off a slice of earth 6 to 9 ins. thick and about 2 ft. wide. The loosened earth is forced to the back part of the scoop, exactly as in loading a drag or wheel scraper, and there it is caught by the buckets of the elevator. This, it appears to us, is an excellent solution of the problem of loading the elevator buckets, for a direct and powerful pull can be applied to the scoop so as to cut through tough soil, and at the same time neither the elevator nor its frame work is put under strain.



A NOVEL TRENCH EXCAVATING MACHINE, NEW ORLEANS, LA.
J. W. Stephens, Inventor.

The upper end of the bucket elevator is supported by pivoted connections to a controlling boom shown in a horizontal position in the photograph. This controlling boom in turn is supported by a bale and a wire rope which passes over a sheave at the top of the crane boom and down to the hoisting drum. While the controlling boom is shown in a horizontal position, its outer end may be raised or lowered, the other end being mounted on trunnions.

In starting a trench the first operation is to raise the controlling boom to such a height as to allow the scoop or digger to swing back as far as possible beyond the vertical; then it is lowered until the teeth of the scoop bite the earth; whereupon the scoop is drawn forward and upward by the cable to the hoisting engine. By slowly hauling the machine forward on its track, and slightly lowering the controlling boom at each swing out, the desired depth of cut is attained. Thereafter straight-ahead cutting is done by raising and lowering the scoop. It should be noted that the machine is always supported on solid ground in advance of the trench. Furthermore, it cuts a trench with perfectly smooth side walls, requiring no hand trimming. In a trench 6½ ft. wide and 12 ft. deep, the three cuts necessary to take off a slice 6 to 9 ins. thick are stated to have required less than two minutes, the actual time being 35 secs. for one of the cuts. The engine used was of 15 HP.; but with a more powerful engine and a wider scoop, even more rapid work would be possible.

The inventor of this machine, on which patents are pending, is Mr. J. W. Stephens, Assistant Engineer, Sewerage and Water Board, New Orleans, La.

THE NEW YORK RAPID TRANSIT RY. during the first 29 days of its operation carried 5,838,235 paying passengers, an average daily traffic of a little over 200,000. This applies of course only to the section of the system first opened extending from the City Hall to 145th St. The heaviest traffic occurred during a few hours of the opening evening when for a time an average of over 22,000 persons per hour were transported. With this record for the opening month and with the system only partially open for traffic it appears probable that within a year when the extension to the Battery and the lines above the Harlem River are open, a traffic approaching half a million passengers daily will be carried on the entire system.

A REMARKABLE LOCOMOTIVE RECORD was made on Thanksgiving day, Nov. 24, by the Cole four-cylinder balanced compound locomotive of the New York Central & Hudson River R. R., described in Engineering News of June 23 last. The record was made on the locomotive testing plant of the Pennsylvania R. R. at the Louisiana Purchase Exposition.

The first test was run continuously for two hours at a speed of 57 miles per hour. The second test continued for an hour and a half at a speed of 66 miles per hour. The third test was for a full hour at the remarkable speed of 75 miles. The drawbar pull was equivalent to a load of five of the heaviest Pullman cars—the number of revolutions per minute was 320. The driving wheels are 6 ft. 7 ins. in diameter. In none of the previous tests of locomotives, which have been made at this apparatus this summer, has it been possible to exceed a speed of 200 revolutions per minute without heating the bearings of some of the parts. The performance of this locomotive is considered very remarkable not merely on account of the speed attained and maintained for so long a time, but because of the smoothness of the running, there being almost no perceptible vibration and the locomotive showing very perfect balancing.

A MUNICIPAL ELECTRIC LIGHTING PLANT FOR Milwaukee was favored by a large majority at the November election this year. A strong fight against such a plant is being made by the local electric company and some of the citizens.

A SMALL WATER-WORKS RESERVOIR at Red Wing, Minn., failed on Nov. 19. The reservoir had a capacity of 1,000,000 gallons and appears to have been built when the works were constructed by the city, in 1883. From a brief description of the works published in this journal for Sept. 6, 1884, it appears that the reservoir was 18 ft. wide, 200 ft. long, 20 ft. deep, excavated in the side of a rock bluff and lined with brick laid in cement mortar. The reservoir was 235 ft. above the pumps, which were located on the Mississippi River. According to a press dispatch it seems that the failure originated in the bottom of the reservoir, but this by no means is certain. The only injury to adjacent property reported is the destruction of a residence.

AN UNDERGROUND FREIGHT RAILWAY system at Chicago is to be established by the Chicago Subway Co., which has been incorporated at Trenton, N. J., with a capital stock of $50,000,000, to operate a freight handling system in the tunnels built by the Illinois Telephone & Telegraph Co. and later transferred to the Illinois Tunnel Co. Mr. A. G. Wheeler, president of this latter company states that $30,000,000 of its stock has been taken by the new company. There are now nearly 30 miles of tunnel 6 × 7 ft., and tunnels 12 × 14 ft. are permitted in the district bounded by Michigan Ave., 12th St. and the Chicago River. The tunnels are about 26 ft. below the surface. The franchise under which the company will operate has a life of 30 years, dating from Feb. 19, 1890. In 1903 it was amended to permit the use of the tunnels not only for carrying the telephone cables, but also for the transmission of newspapers, mail, parcels and freight, but their use for passenger service is expressly prohibited. The purpose of the new arrangement is to interest the railway companies in the development of a new system for handling freight, as explained in the following statement issued by Mr. A. G. Wheeler, who will continue in charge of the operation:

> The tunnels finished under the streets, to prove of great benefit to the community, must have the traffic diverted from the streets and transported through the tunnels. While no question was ever in any one's mind that ultimately this would be accomplished, yet to have the community reap the benefit of the removal of such congestion at once it became necessary that the parties controlling the steam railways with terminals in Chicago should be interested in the tunnels, and immediately use the tunnels for the transfer of the freight from the railroad terminals to the basements of the business houses. To produce this result and get the tunnels in full operation at the earliest date, the holders of two-thirds of the stock of the Illinois Tunnel Co. sold their stock to a corporation called the Chicago Subway Co., controlled by the owners of all of the steam railways that terminate in Chicago, and represented by E. H. Harriman and James Stillman, of the National City Bank, New York City; Kuhn, Loeb Co., through Jacob H. Schiff; and P. A. Valentine, of Chicago, who is the local representative of the railway interests.
>
> The operation of these tunnels, now under the control of the representatives of the railway interests terminating in Chicago, will prove beneficial to all merchants handling freight in and out of their buildings, and to the general public in having the streets relieved of congestion. The broad scope of the development of these tunnels through this representative railroad interest cannot help being a great commercial advantage to the future growth of the city. The Chicago Subway Co. has no connection with, nor are any parties interested in it connected with, the street railways of Chicago.

SEWAGE WORKS FOR GLENELG, SOUTH AUSTRALIA, are described by our Adelaide correspondent as follows: The sewage is pumped some 50 ft. in elevation from a concrete well to a catch-basin (grit chamber?—Ed.). The pumping machinery consists of two 25 B. HP. Crossley oil engines and two belt-driven centrifugal pumps of American type, having 8-in. suction and delivery pipes. One engine and pump is held in reserve. From the catch-basin the sewage is discharged through gates into three septic tanks, 15 × 75 ft. in plan. The septic tank effluent is discharged periodically, through hand-operated gates, into a dosing tank placed alongside the ends of the septic tanks. The effluent is siphoned from this tank and distributed on the four filter beds in rotation, the distribution being controlled automatically by a float which in rising gives, by means of a pawl, a quarter turn to a shaft carrying four cams which operate the distribution levers for the four filter beds. The drifting nature of the sand where the works are situated necessitated the planting of couch grass and rushes to retain the ground. The work has been carried out to the designs and under the supervision of Mr. C. A. Bayer, of Adelaide.

THE STANDARD RAILS of leading English railways are as follows: (1) Midland Ry., 100 lbs. per yd., 36 ft. long; (2) Great Western Ry., 97½ lbs., 30 ft.; (3) Great Northern Ry., 96 lbs.; (4) Great Central Ry., 96 lbs., 36 ft., and 86 lbs., 30 ft. long; (5) London & Northwestern Ry., 90 lbs., 100 ft.; (6) Northeastern Ry., 90 lbs., 30 ft.; (7) London & Southwestern Ry., 90 lbs; (8) Lancashire & Yorkshire Ry., 86 lbs.; (9) Great Eastern Ry., 85 lbs. All of these are of the bull-head section, supported in heavy cast-iron chairs, to which they are secured by wooden wedges (or sometimes wedges of coiled steel). The chairs are fastened to the ties by round spikes and wooden spikes or treenails, through bolts being occasionally used.

THE EIGHT-HOUR LAW OF NEW YORK STATE has been declared unconstitutional in its entirety by the Court of Appeals, which is the court of last resort. There have been three previous similar decisions, each on some particular section of the law. The state and municipalities in the state can therefore no longer require contractors to work their employes only eight hours per day.